1958

Effect of hydrated lime and portland cement on the physical properties of clay soil for highway base construction

Judson Leong

EFFECT OF HYDRATED LIME AND

PORTLAND CEMENT ON THE PHYSICAL PROPERTIES

OF CLAY SOIL FOR HIGHWAY BASE CONSTRUCTION

By

JUDSON LEONG

A

THESIS

submitted to the faculty of the

SCHOOL OF MINES AND METALLURGY OF THE UNIVERSITY OF MISSOURI

in partial fulfillment of the requirements for the

Degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING

Rolla, Missouri

1958

APPROVED:

Major Advisor

Chairman of Department

ACKNOWLEDGMENT

The writer wishes to express his grateful appreciation to Professor John B. Heagler, Jr., of the Civil Engineering Department, Missouri School of Mines, for his invaluable advice, guidance and constructive criticism throughout the course of this work.

The writer is indebted to Professor Ernest W. Carlton, Chairman of the Civil Engineering Department, Missouri School of Mines, for his encouragement and review of this manuscript.

Appreciation is also expressed for cooperation given by the various staff members of the Civil Engineering Department, Missouri School of Mines.

Acknowledgment is due Mr. Joe Pokorny of the Illinois Division of Highways, Peoria, Illinois, for acquainting the writer with soil stabilization problems in highway construction.

TABLE F CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

INTRODUCTION

The problem of building suitable roads has existed ever since the invention of the wheel and men will face this problem as long as civilization exists. One of the many problems which confronts the highway engineers who are responsible for the design and construction of highways, whether they are super highways or farm-to-market roads, is to provide an adequate foundation.

In order to appreciate the importance of the subgrade in road construction, it must be pointed out that the ground or subgrade beneath the pavement or wearing course really supports the load of the moving vehicles. If a pavement or a wearing surface is to function satisfactorily, it must have either a sound stable subgrade, or a thick and stable enough base course must be provided so that loads imposed by traffic on the pavement or wearing course may be transferred to the subgrade without overtaxing the supporting ability of the soil.

The functions of a pavement or wearing course are: to distribute the wheel load and impact of moving vehicles over areas larger than those furnished by the tire alone so as to prevent deformation which would be detrimental to either the road surface or the subgrade; to serve to resist the wear and tear caused by traffic; and to shed a large portion of rain water which falls on the surface.

For maximum stability, resistance to deformation and strength, subgrade soils must have the proper proportions of aggregate and soil binder. Theoretically, stable mixtures consist of well graded coarse material possessing high internal friction and a binder. The binder which may be visualized as occupying the sand pores, should have sufficient cohesion to cement the sand grains together. Upon wetting, the binder should expand in an amount just sufficient to close the pores and thus prevent water from penetrating and softening the interior of the road surface.

In some localities, the natural soils are suitable to provide adequate support for pavement or wearing course. However, in many areas, soil in its natural state is too weak to support the load. In such cases, an adequate foundation may be provided by stabilizing the soil through the use of suitable natural or chemical materials, or by providing a base course, or both. The method selected for stabilization of roadbeds is dependent upon local economics of road construction. In many cases, suitable natural material for admixing will not be economically available; therefore, some other type of stabilizing agent such as lime, cement, asphalt, etc. will have to be considered.

Enormous expansion in the highway construction program has emphasized the importance of improving by some means, locally available material which would otherwise be unsuitable for modern highway construction. The problem of good road material is becoming more serious and areas possessing adequate materials are facing a serious depletion due to the heavy consumption of the materials.

One of the factors contributing to the economic growth of any country is transportation. Of the different types of transportation, highway transportation has been recognized as the most vital of all. Many so called "under-developed" countries such as the writer's native land, Burma, are launching economic expansion, and industrial development programs. In order to carry out these programs effectively, the existing highways must be improved and expanded to meet the need.

The existing highways in Burma were designed and built for light traffic. Most of the roads in Burma are gravel, untreated macadam and bituminous surfaced roads. Since there is very little heavy traffic at the present time, the construction of high type pavement cannot be justified. Also, the economic condition in Burma will prevent improvement to a higher type of road in the near future. Until the time when these roads can be built up to a higher type, they can be most economically improved by soil stabilization.

Limestone deposits have been found in many parts of Burma. All along the sea coast, sea shells, which are almost pure calcium carbonate, are abundant. These materials can easily be converted into hydrated lime. Because of the availability of suitable raw material for the manufacture of portland cement, the government is planning to construct several new cement factories. Thus, hydrated lime and cement will be available for highway construction.

It is the realization of the above factors that prompted the writer to conduct this research.

The intent of this research is to determine the change in physical properties of a clay soil caused by the addition of 2, 4, 6, 8 and 10 percent of hydrated lime and portland cement. The effects of these admixtures on the physical properties of the soil were observed by comparing the results of various laboratory tests. The properties chosen for comparison were: (1) liquid limit, plastic limit and plasticity index; (2) shrinkage limit and shrinkage ratio; (3) density and optimum moisture; (4) resistance to penetration; (5) compressive strength and (6) cohesion and angle of internal friction. In order to compare the effect of freezing and thawing on specimens containing hydrated lime and cement, specimens of various soil-lime and soil-cement mixtures were subjected to 6 freeze-thaw cycles and the influence of these admixtures observed.

REVIEW OF LITERATURE

The use of lime in road construction dated back to the Romans who built the Appian Way. They used rock of various sizes, sand and lime for construction materials. The Appian Way consists of 5 layers, and in 3 layers lime was used.

It was also reported that the Chinese have used a crude form of lime stabilization on the rural and village roads of China for years. Lime was simply mixed with soil in place and compacted by crude methods.

In the United States the earliest experiments using hydrated lime for road construction was in 1923, when the Engineering Experiment Station of the University of Missouri, in cooperation with the National Lime Association, treated 500 ft. of clay road with hydrated lime near Hallsville, Missouri(1). It was an attempt to determine the effect of

(1) McCaustland, E. J., Lime in Dirt Road, Pit and Quarry, Vol. 10, No. 5, pp. 93-95, June, 1925.

the use of lime on earth roads with the hope of preventing people from getting "stuck in the mud". The result was favorable, for it was reported that "the clay and lime mixture does not stick on the wheels of passing vehicles but smooths out and packs much more quickly than does the untreated clay". This was the first report concerning the possibilities of using lime treatment to stabilize road subgrades.

From that time until about 1938, nothing of consequence was attempted with lime for road stabilization. In 1938, The Texas Highway Department conducted some laboratory tests with varying percentages of lime on different types of soils.

Although many articles describing the construction and performance of lime stabilized roads are available, there is very little published information concerning laboratory experiments on lime-soil mixtures.

The results of laboratory research conducted by the Engineering Experiment Station of Purdue University, sponsored by the National Lime Association, was reported by Mr. A. M. Johnson[2] during the twenty-eighth annual meeting of the Highway Research Board in 1948.

(2) Johnson, A. M., Proceeding, Highway Research Board, Vol. 28, pp. 496-507, 1948.

The first part of the report describes the Atterburg limit tests on 25 fine-grained soils, with no admixtures, and with 2 and 5 percent of hydrated lime. In each case, lime was mixed dry with the soil, and test was begun immediately after the addition of water to the mixture. It was found that soils having plasticity index less than 15 increase in their plasticity indices with the addition of lime. On the other hand, soil with plasticity index more than 15, their plasticity indices decrease with the addition of lime. If a division were to be made between silt and clay soil on the basis of plasticity index, with plasticity index 15 as the dividing line, it can be stated that plasticity indices of silty soils were increased with the addition of 2 or 5 percent of hydrated lime. But plasticity indices of clay soils were lowered considerably by lime additive.

In order to determine whether time of standing would have any effect upon the plasticity index or change the effect of lime added, 16 of the soils were tested at 7 and 14 days after water was added to the raw soil or to the lime-soil mixture. It was observed that by allowing the soil to stand for a period of time after wetting tends to cause a slight increase in plastic index. This trend was apparent with the raw soil as well as with that lime-soil mixture.

Compaction tests were made on 11 soils and lime-soil mixtures, using standard Proctor procedure. The plastic indices of these soils varied from 3 to 27 and the densities from 89.4 to 113.9 lbs. per cu. ft. Penetrometer readings were taken on all compacted specimens. Although the results were not consistent, there was a general trend of reduction in maximum dry density accompanied by a slight increase in optimum moisture content. From penetration test, it was found that the addition of lime increased the resistance to penetration, even though the density is less.

Nine of the eleven soils used evidenced increase in resistance to penetration resulting from the use of lime at all moisture contents used. The remaining two soils showed some variation at lower moisture contents but appeared stronger with lime at optimum moisture content.

Tests in part 2 were performed on 5 natural gravels. Optimum moisture content of these soils were determined using a modified procedure and CBR molds. CBR specimens were molded at or near optimum moisture content with 0, 2 and 5 percent of lime. CBR tests were made of each combination of soil and lime, following three different methods of curing: (1) as molded; (2) after seven days capillary saturation; and (3) after drying to constant weight at 140°F and seven days saturation. Addition of lime produced increased strength in most specimens. In most cases, the period of drying before wetting added to the strength at the time of testing if lime had been added to the soil.

The synthetic gravel-binders were used in part 3. Mixing, curing and testing procedures were the same as in Part 2. Increase in strength, as measured by the CBR tests, was also observed.

The results of the same project was reported by Professor K. B. Woods[3] at the thirty-first Annual Convention of the National Lime

(3) Woods, K. B., Lime as an Admixture for Base and Subgrades, Paper presented at the 31st Annual Convention of the National Lime Association, 1949.

Association, with the addition of a series of unconfined compression test on fine-grained soils. Specimens were prepared in the Proctor mold in the standard manner, compacted at the respective optimum moisture contents determined in the compaction study. Specimens were prepared with soil alone, and with 2 and 5 percent lime added. Three series of unconfined tests were run. In one series, thespecimens were tested immediately after molding. In the second series, they were removed from the molds and placed on porus discs which permitted water to rise by capillary action through the specimens. The specimens were maintained in that condition for seven days after which they were tested. In the third series, the specimens were taken out of the molds and were placed in an oven at 140°F. for seven days, after which they were placed on porous discs in water to permit wetting by capillary action. The specimens were loaded in a testing machine until failure occurred. The test results indicate that, as far as fine-grained materials are concerned, subjected to the above curing procedures, there was a marked increase in strength with the addition of 2 and 5 percent lime.

In comparing curing conditions, the drying was effective in increasing the strengths of specimens containing 5 percent lime. The drying out produced a change even after the soil was wetted again.

In the same paper Mr. Woods[4] also presented the results of a

(4) Woods, K. B., ibid., p. 4.

series of unconfined compression test for the purpose of investigating the effect of different brands of lime on the strength of compacted soil. Four brands of lime; two dolomitic lime and the other two calcium lime. It was observed that the specimens with an admixture of calcium lime exhibited a higher strength than those prepared with dolomitic lime.

Mr. C. McDowell and Mr. W. H. Moore(5) of the Texas Highway

(5) McDowell, C. and Moore, W. H., Improvement of Highway Subgrades and Flexible Bases by the Use of Hydrated Lime, Proceeding of the Second International Conference on Soil Mechanics and Foundation, Vol. 5, pp. 260-267, 1948.

Department conducted some experiments on lime-soil mixtures by unconfined compression and triaxial tests. Seven soils with plasticity indices varied from 18 to 45 were prepared, followed by 7 day moist-curing. Specimens prepared with soils whose plasticity indices of less than 30 were dried at 140°F, but those prepared with soils whose plasticity indices of greater than 30 were air-dried partially, since complete drying might cause these specimens to crack. Curing was followed by capillary wetting,some for 10 days and some for 30 days. No mention was made as to the moisture content of the specimens at compaction. The ultimate unconfined compressive strength of the specimens were compared with the ultimate strength of an untreated crushed rock specimen which was considered good flexible base material. Results showed that the ultimate compressive strength of untreated specimens were below the ultimate compressive strength of the crush rock specimen. But the ultimate compressive strength of all the lime treated specimens were much higher than that of the crushed rock specimen. Percentages of lime used ranges from 3 to 9 percent. This promising result led them to investigate further using triaxial tests.

In this triaxial test series, specimens were made with two different compactive effort; the standard Proctor compactive effort of 6.63 ft. lbs.

per cu. in., and 13.26 ft. lbs. per cu. in. Correlation of field density tests with laboratory compaction procedure showed that a compactive effort equivalent to 13.26 ft. lbs. per cu. in. of specimen will compact specimens of flexible base material to the density usually found in finished construction. A wide variety of soils, including crushed rock, clay-gravel, sand-clay, sandy soil, medium and heavy clay soil were used. Their plasticity indices ranges from 6 to 45. Specimens compacted with compactive effort of 13.26 ft. lbs. per cu. in. showed an increase in strength of 25 to 85 percent. McDowell and Moore concluded that densification is of critical importance. Only one Mohr's diagram of stress was shown as an example. The following were the conclusions drawn by McDowell and Moore.

(1) Soil-lime stabilization has a definite application in highway construction for the improvement of certain subgrade and flexible base material.

(2) Many natural soils are suited to lime stabilization. The identical materials proposed for use should be subjected to preliminary physical tests.

(3) Good proportioning and mixing of constituents are advantageous.

(4) Compacting moisture should be at, or slightly below, optimum moisture content for the compactive effort employed.

(5) A high degree of compaction is of critical importance.

(6) Suitable curing procedures are important.

(7) Application of a wearing surface is desirable.

Experiments on durability of lime-stabilized soil, as determined by freezing and thawing tests, were performed by E. A. Whitehurst and E. J. Yoder(6) on three soils, a Wisconsin drift soil, an Illinois drift soil

(6) Whitehurst, E. A., Yoder, E. J., Proceedings, Highway Research Board, Vol. 31, pp. 529-540, 1952.

and a river terrace gravel, with the addition of 0, 2, 5 and 10 percent lime. The purpose of their project, as stated by Whitehurst and Yoder, were "to determine the durability characteristics of lime-soil mixtures as affected by such variables as soil texture, soil density and quantity of lime; to determine the effect of moist curing on the unconfined compressive strength and durability of lime-soil mixtures; and to explore further the suitability of dynamic testing techniques for evaluating the performance of such mixtures".

Standard classification tests, including the Atterburg limits, mechanical analysis and Proctor compaction were made on representative samples of each soil. Specimens were molded in split cylinder Proctor molds at optimum moisture content as determined by compaction tests. The quantities of lime used were 2, 5 and 10 percent by dry weight. These specimens were allowed to moist-cure for periods of 1, 4, 8, 15 and 36 weeks. Duplicate specimens were made in all cases. At the end of the curing period one of these was broken in unconfined compression to determine the effect of lime upon strength. The other was subject to freezing-and-thawing, a 48 hour cycle being employed, until it fell apart or until 12 cycles had completed. The specimens which underwent freezing-and-thawing were tested at the end of each cycle with the soniscope. The characteristic measured by the soniscope is the velocity of pulse propagation through the test specimen, which is a measure of rigidity, dynamic modulus of elasticity, etc.

From the investigation, the following conclusions were drawn by Whitehurst and Yoder.

(1) The texture of the soil has an appreciable effect upon the resistance of the lime-soil mixtures to freezing and thawing. The soil-aggregate mixtures appear to have considerable promise. It is indicated, however, that greater care must be exercised in achieving thorough distribution of lime and moisture and proper compaction with these materials than with the fine-grained soils.

For a given lime content, increased compaction, or greater density, result in increased resistance to freezing and thawing.

Lime in quantities of 5 percent, or more, by weight, greatly increased the durability of the lime-soil mixtures, the greater the lime content the greater durability; 2 percent lime did not appreciably alter the durability characteristics of the soil.

(2) In general, moist curing proved very beneficial to the lime-soil mixtures. There is evidence, however, that when the fine-grained soil mixtures were exposed to 100 percent humidity for lengthy periods, they tended to take up moisture in detrimental quantities.

(3) The dynamic test employed in this study, the measurement of pulse velocities through the test specimens, was quite satisfactory. Results were reproducible and there appeared to be little operator error. It is believed that changes in velocity are highly indicative of changes in the quality of specimens tested.

The effect of lime on the cohesion of soils were tested by Mr. R. F. Dawson[7] using Hveem Cohesiometer. A red clay gravel known to exhibit large volume changes, with liquid limit ranges from 50 to 65 percent and plasticity index ranges from about 25 to 40 was used in this experiment.

---

(7) Dawson, R. F., Special Factors in Lime Stabilization, Highway Research Board, Bulletin 129, pp. 103-110, 1956.

The 6 in. diameter specimens with various percentages of lime added were compacted with two different compactive efforts: namely, 6.63 ft.-lbs. per cu. in. and 13.26 ft.-lb. per cu. in. The compacted cylinders were placed under a one-psi. all-around pressure and permitted to saturate by capillarity through a porous stone. Curing time ranges in age from zero up to four months. Specimens were tested immediately on removal from the moist room. From the test data, it was found that cohesion increases (1) as the time of curing increases, (2) with the increase in compactive effort, and for this particular soil, 5 percent of lime gave the highest cohesion. Dawson emphasized om the benefit of long curing by saying, "tests for accelerated tensile strength, such as the cohesionmeter test, have traditionally been unrealistic for lime stabilization in the construction field unless some unusually long periods were used. While the results obtained here show a considerable increase in strength up to a period of four months, they also indicate that the strength increase would be expected to continue far beyond this period; and it might well be expected that curing period of six months to a year or even longer would give much higher strength than those currently indicated. The increase in strength with age is due to the fact that lime gains in strength through pozzolanic action and that carbonation takes place slowly".

The stabilizing reaction of lime with clay soil was studied by Mr. B. M. Gallaway and Mr. S. J. Buchanan[8]. Of the three groups of clay

(8) Gallaway, B. M. and Buchanan, S. J., Lime Stabilization of Clay Soil, Texas Engineering Experiment Station, Bulletin No.124, 1951.

minerals; namely, montmorillonite, illite and kaolinite, the montmorillonite group is believed to have the most surface activity. Therefore, Gallaway and Buchanan seek a positive method of identifying the montmorillonite. Four methods were experimented, and the following conclusions were made:

The results of the differential thermal analysis was indicated to be satisfactory. However, the results of such identification were indicated to be of general, rather than specific value, as regards lime stabilization. It did not appear possible to identify the sodium cation from the calcium cation montmorillonite clays by this method.

The identification of members of the clay family by the x-ray method is possible, but at present it does not appear to be feasible for engineering purposes.

The unreliability of the benzidine color reaction test for identification of montmorillonite was confirmed.

The results of the determinations of total base exchange and plasticity characteristics were studied by Gallaway and Buchanan so as to ascertain whether or not a correlation could be established between the two. A correlation is found to exist by plotting the plasticity indices as the ordinate to a logarithmic scale and the total base exchange plotted as the abscissa to an arithmetic scale. Gallaway and Buchanan states that, "the correlation of the plasticity index with the base exchange, as developed for both laboratory and natural soils, appear to be reasonable and permits ready identification of clay soils which would react favorably with lime as a stabilizing admix".

Apparently the admixing of cement with soil in road construction was not tried until 1932, when South Carolina State Highway Department experimented with a soil-cement mixture hoping that it might be used as a base material for light traffic roads(9). Mixtures of portland cement

(9) Mills, W. H., Road-Base Stabilization with Portland Cement, Engineering News-Record, Vol. 115, pp. 751-753, Nov. 28, 1935.

with top soil and with sand were molded into pats 8 in. in diameter by 1 in. thick. On exposure to the weather it was found that the cement top soil mixtures resist disintegration from rain much better than soil alone or the cement-sand mixture. Subsequently, soil-cement blocks, 2 ft. square by 8 in. thick were made on a driveway to study their resistance to weather and traffic. The quantity of cement used was 1, 2, 3 and 4 bags per cubic yard of soil. The mixing was "done by hand and sufficient water was added to make the mixture plastic". Ten months later, it was reported that "all show wear apparently in inverse proportion to the amount of cement they contain". Small blocks were cut from each specimen, and boiled for 15 hours. It was observed that "no softening or disintegration occurred, but the soil without the admixture of cement softened rapidly on exposure to steam".

The first field experiment was constructed in December, 1933. A section of sand clay soil in place was pulverized, and cement applied to the surface at one bag per linear foot of 20 ft. wide roadway(10). Soil

(10) Mills, W. H., Stabilizing Soils with Portland Cement, Experiments by South Carolina State Highway Department, Proceeding, Highway Research Board, Vol. 16, pp. 322-349, 1936.

and cement were mixed dry, sprinkled, mixed wet, shaped and rolled. A few pot-holes were reported after a year of service, but there was no indication of raveling or general breakdown.

Before the construction of another experimental road section, an intensive laboratory study of soil compaction and of resistance of soil-cement mixtures to repeated wetting and drying, freezing and thawing,were made. It was reported that "with one exception the soil-cement mixtures have higher densities, or dry unit weights, than raw soil, and that there is a slight decrease in the optimum moisture content producing maximum density".

Durability tests were of two types: (1) The specimens were first subjected to repeated wetting and drying and then (2) subjected to repeated freezing and thawing which also included wetting and drying. From the laboratory tests and experience on previous experiments, it was decided to use 6 percent of cement in most of the field work plus one percent for loss in placing and mixing. In addition to these tests, a series was conducted to determine the possibility of adding cement in the form of "slurry". The required quantity of cement and water were mixed and then this mixture was combined with the dry soil. It was noted during the mixing of "slurry" and soil and again after this mixture was compacted in the mold that small balls of cement varying in size from 1/8 to 1/4 in. diameter had formed. Because of the formation of cement balls, it was decided not to try a field experiment by this method..

The test section was constructed in the summer of 1935. A temporary wearing course of cut back and sand was applied soon after the base was completed. The section was inspected during the Spring, and was reported "to be in good condition, with no failures except in two small poorly constructed areas where cracking and small pot-holes had developed". Tests of cores from this section indicated that mixing of cement was fairly uniform. Average compressive strength at 86 day was found to be 480 pounds per square inch. Mills also states that "durability tests clearly indicated the benefit of adding cement to raw soil".

The same method of soil stabilization was applied to another 2 miles of road during the summer of 1936. The base was reported to be in excellent condition at the time a wearing course was applied in July, 1936.

Mills did not attempt to draw any definite conclusions at that time. He states that, "the action of weather and traffic will in time evaluate the worth of this method of stabilization. The present indication is that treatment of soils with portland cement has appreciable merits and is possible and comparatively economical for many light traffic roads in South Carolina".

Portland Cement Association began laboratory research on soil-cement in January, 1935(11).

(11) Sheets, F. T. and Catton, M. D., Basic Principles of Soil-Cement Mixtures, Engineering News-Record, Vol. 120, pp. 869, 875, June 23, 1938.

A wide range of soil types from various parts of the United States were obtained, and their physical test constants and grain size were determined. Then their identifications were made. Moisture-density relations of the raw soils were conducted using standard Proctor method of compaction. The influence of various percentages of portland cement on moisture-density relations of the soil were determined using standard Proctor method. In order to determine the influence of various cement contents on the durability and stability of soil-cement mixtures, specimens with different percentages of cement were compacted at optimum moisture using the standard Proctor method. Specimens were air-dried in moist atmosphere for 7 days, brushed with wire brushes to remove loose material and immersed in tap water for 5 hours. The wetting and drying was continued for 12 cycles, and at the end of each cycle, the weight of the specimen was determined.

After 7 days moist curing two other specimens of each mixture were placed in a refrigerator subjected to 20 hours freezing. Then the specimens were allowed to thaw in a moist room for 24 hours, with free water added to pads as needed to permit capillary absorption by the specimens. The specimens were then wire brushed to remove loose material and established soil loss. All specimens were subjected to 12 cycles of freezing and thawing.

As durability tests gave results that reveal definitely the influence of cement on the durability of soil-cement mixtures, Sheets and Catton divided the soils tested into three general groups. Soils showing a very marked hardening, as determined by durability tests, with addition of cement were placed in Treatment Group I, soils showing marked hardening, with the addition of cement were placed in Treatment Group II, soils showing substantial hardening with the addition of reasonable amount of cement were placed in Treatment Group III. Another group of unusually bad soil, whose optimum-density curves were different from the curves of the other soils were placed together in Group IV.

Sheets and Catton summarized these grouping together with the test constants, thereby showing a direct correlation between the hardening influences of cement on soil-cement mixtures and soil characteristics. Sheets and Catton further states that, "as data of this nature is obtained from other soils and soil-cement mixtures and added to the tabulation, more exact relation will be set up between the hardening influence of cement and soil characteristic and thus permit predetermination of treatment requirements without recourse to detail durability tests".

An extensive research on the physical relations of soil and soil-cement mixtures were conducted by Portland Cement Association[12]. Tests

(12) Catton, M. D., Research on the Physical Relations of Soil and Soil-Cement Mixtures, Proceedings, Highway Research Board, Vol. 20, pp. 821.

on 329 soils from 37 states were performed by the Portland Cement Association. Included were the usual routine tests on soils and moisture-density, wet-dry, freeze-thaw and compressive strength determinations on soil-cement mixtures. These tests showed the predominate physical relations of soil and soil cement mixtures. A grouping of soils according to the United States Public Road Administration classification shows, in general, that in the A-2 and A-3 groups will require 6, 8 or 10 percent cement by volume for satisfactory result, the A-4 and A-5 group will require 8, 10 or 12 percent cement by volume and the A-6 and A-7 group will require 10, 12 or 14 percent cement for satisfactory results. These same data show the generality that cement requirement increase with silt and clay content. Data on hydrogen ion concentration (pH) were included, showing that a soil may be acid, neutral or alkaline and yet it will respond satisfactorily to hardening with cement. The study of organic contents of soils showed this factor to be most diverse and it has a variable influence on soil-cement mixtures ranging from no preceptible to a major influence.

Catton pointed out that such factors as grain size, gradation, silt and clay content, density, optimum moisture, water holding capacity, surface area, organic content, void-cement ratio, hydrogen ion concentration, compressive strength, etc., contribute to an analysis of soil and soil-cement relation, but they are so diverse and interrelated in character and influence, that none of them have a constant, major predominating influence. Catton further states that all these factors together show that some factor or influence of a chemical or physio-chemical nature, such as the mineral composition of the soil grain and its absorbed ions, may play a predominate part in evaluating soil and soil-cement relation. He stressed that research on these factors will contribute valuable information on these relations.

Research on various factors influencing physical properties of soil-cement mixtures was carried out by Portland Cement Association[13].

(13) Felt, E. J., Factors Influencing Physical Properties of Soil-Cement Mixtures, Highway Research Board, Bulletin No. 108, pp. 138-162, 1955.

A large variety of soils were obtained and nine series of tests were made to determine the influence of various factors upon the compressive strength and resistance to wetting, drying, freezing and thawing of compacted, hydrated soil-cement mixtures. It was discovered during this research that most of the soil-cement mixtures, when compacted according to standard compaction test, develop parabolic-shaped moisture-density curves and a maximum density is indicated at an optimum moisture content at the peak of the curve. But for clay soil-cement mixtures, because of their swelling characteristics as they become wet, an irregular moisture-density curves of ski-slide shape was formed. The base-line optimum moisture content of such soil-cement mixture was generally taken about 2 percentage points above the water content at the second hump.

Preliminary tests using standard wet-dry and freeze-thaw test were made to determine the cement requirement for each soil.

In series 1, the effect of density on the quality of soil-cement was investigated, the quality being the evaluation of the results of wet-dry, freeze-thaw and unconfined compression tests. In order to vary the density of the specimens, number of blows during compaction were varied. All three methods of evaluation showed that the quality of the soil-cement showed marked improvement with the increase in density. Although all the different types of soil-cement mixtures were benefited by increased density, the silty and clayey soil-cement mixture showed the most benefit by increasing the density.

In Series 2, the effect of molding moisture content on the quality of soil-cement mixtures was studied. Specimens for wet-dry, freeze-thaw and compression tests were compacted below and above the selected base-line moisture content using standard method of compaction. Since the compactive effort was constant and the moisture content varied, the density of the specimens also varied. Felt explained, "the data indicate that the effect of moisture content overshadows the effect of the differences in density". The wet-dry, freeze-thaw, and compressive strength data, when considered together, indicate that for maximum effective strength from the cement-sandy soil, the mixture should be compacted at optimum moisture content or slightly drier, whereas silty and clayey soil-cement mixtures should be compacted at moisture content 1 or 2 percentage points above optimum moisture.

Series 3. During construction, damp mixing of soil-cement may continue for 2 hours or more. This series was a laboratory study of the effect of prolonged damp-mixing on the soil-cement mixtures. Soil-cement was damp mixed for periods of 2, 4 and 6 hours and molded into test specimens. Water was added to the dry mix in equal increments of 20 minute intervals. After each addition of water, the mixture was stirred for about 2 minutes. Wet-dry, freeze-thaw and compressive test specimens were molded at optimum moisture content using standard AASHO compaction method. Data show that optimum moisture content increased and the maximum density decreased as the length of mixing time increased. Wet-dry and freeze-thaw test data for these soil-cement specimens show that the losses of weight increased as the length of the damp-mixing period increased. Although results of compressive tests in this series is consistent in most cases, strengths decreased with the time of mixing.

In Series 4, investigations were made in order to determine if the degree of pulverization effects the quality of soil-cement mixture.

specimens were compacted at optimum moisture content, each containing 0 percent, 20 percent and 40 percent lumps retained on a No. 4 sieve but passing a 1 inch sieve. In one set of specimens (A), air-dry clay lumps were added to the minus No. 4 mixture which was at optimum moisture content. Specimens were molded immediately. In the second set (B), air-dry clay lumps were added to air-dry minus No. 4 material. Water than added to the total mix to bring it to optimum moisture content. Thus, in set A, the clay lumps had less opportunity to absorb moisture during the mixing period than in Set B. It was found that specimens of Set A had less resistance to alternate freezing and thawing and dry than Set B. Felt stated, "In some cases complete failure occurred by disruption of the specimens of Set A as the dry clay lumps absorb water and swell during the curing and the testing periods. When the clay lumps were damp (Set B) and thus swelled condition at the time of inclusion in the test specimens, the unpulverise soil had little harmful effect".

In Series 5, a study was made to determine the comparative performance of mixtures made with air-entraining and non air-entraining cement. Data from wet-dry and freeze-thaw tests showed there was relatively little difference in test data for the two cements. Compressive strength data also shows minor differences between the strengths obtained with two types of cement.

Series 6 were made to determine the effect of cement content on the quality of the soil-cement mixtures. Specimens were compacted at optimum moisture content, with cement content of 6 to 34 percent. Specimens were subjected to 96 cycles of wet-dry and freeze-thaw tests. The compressive strength and resistance to wetting and drying and freezing and thawing increased as the cement content was increased. Depending upon the soil, but generally good quality mix was obtained with cement contents in the range of 8 to 14 percent. Mixtures having unusually high compressive strength and excellent resistance to alternate freezing and thawing and wetting and drying were obtained with relatively high cement content of about 22 to 30 percent by volume.

In Series 7, compressive strength tests were made to study the effect of high-early strength (Type III) cement in soil-cement mixtures. Specimens of sandy and silty soils, each with 6, 10 and 14 percent of cement were molded at optimum moisture content and maximum density. Optimum moisture content and maximum density for mixtures containing Type I or Type III cement are found to be practically the same. Specimens were broken at ages of 1, 2, 3, 4, 6, 7, 10, 14, 28 and 60 days. For both soil types the early age strength were consistently greater for Type III than for Type I, and in nearly all cases the 60 day strengths were also greater for Type III.

Mixtures of soil with a small quantity of cement added in order to reduce the extent to which the soils shrink, swell and lose strength, are called cement-modified soils by Felt. In Series 8 and 9, the influence of various cement contents in altering the properties of fine-grain and granular soils were studies.

In Series 8, samples of clay soils for the determination of test constants and grain size were compacted at optimum moisture content. These specimens were cured for 7 days at 100 percent humidity. Part of these specimens were pulverized to pass a No. 10 sieve for hydrometer analysis, and part was pulverized to pass a No. 40 sieve for determining test constants. The results of soil constant tests showed that cement effectively reduces the plasticity index and increases the shrinkage limit of clay soils. Resistance to penetration was tried on a compacted cement-modified soil specimen and cured for seven days at 100 percent relative humidity. Specimens were compacted three layers, using 56 blows of the 5.5 lb. hammer on each layer. Cement-modified soil specimens showed a great increase in resistance to penetration.

In Series 9, the effect of additions of relatively small quantities of cement to granular materials was investigated. Two methods of tests, namely, thepenetration test and the soniscope test on specimens at different ages and after various cycles of alternate freezing and thawing, were used. Three granular soils containing various percentages of silt and clay were used in these tests. From the results of soil constant tests, great reduction in plasticity was observed. The specimens for penetration and soniscope tests were retained in the molds and placed in the moist room. Several specimens of each cement content were molded so that they could be tested after various ages in the moist room and after various cycles of alternate freezing and thawing. Thus, the tests permitted a study of the effect of cement content and time of curing on penetration and pulse velocity and also a study of the deteriorating effects of freezing and thawing. Even with relatively low cement content, a high value in resistance to penetration was indicated from the test results. Freezing and thawing reduced bearing resistance for the mixture containing 1.5 percent of cement; but mixtures containing 3 percent or more of cement showed no deterioration during the freeze-thaw tests. The pulse velocity as indicated by soniscope did not decrease significantly during the freeze-thaw test, again showing good resistance to deterioration. The pulse velocities for soil cement mixture increased with increased cement content and with time of moist curing.

MATERIALS

Materials used in this research were soil, hydrated lime and cement.

Soil: All the soil used in the experiments was obtained from a farm owned by Mr. John Heagler, Sr. located about seven miles southeast of Rolla on Highway U. S. 72. The soil is from the B horizon, and may be classified as a reddish yellow Podzolic soil. The reason for selecting the soil from the B horizon is clearly stated by Mr. M. G. Spangler(14).

(14) Spangler, M. G., Soil Engineering, International Textbook Company, p. 29, 1951.

He states "this lower horizon usually contains finer-grained material and often is much more surface-chemically active and unstable than the soil either above it or below it. These characteristics render the B horizon extremely important in highway and airfield design and construction or other work in which the foundations are located near the ground surface".

The specific gravity of the soil was determined in accordance with the Standard Method of Test for Specific Gravity of Soils, ASTM Designation: D854-52(15), and was found to be 2.60.

(15) ASTM Standard, Part 3, American Society for Testing Materials, pp. 1786-1788, 1955.

A liquid limit test conducted in accordance with the tentative method of test for liquid limit of soils, ASTM Designation: D423-54T(16),

(16) ASTM Standards, ibid., pp. 1769-1773.

and was found to be 35.5%.

The plastic limit was determined in accordance with the Tentative Method of Test for Plastic Limit and Plasticity Index of Soils, ASTM Designation: D-424-54T(17), and was found to be 19.0%.

(17) ASTM Standards, op. cit., pp. 1774-1776.

Plastic Index = 35.5 - 19.0 = 16.5

Grain soil analysis of the soil was performed in accordance with ASTM Designation: D422-54T[18] and the grain size accumulation chart was

(18) ASTM Standards, op. cit., pp. 1756-1766.

plotted in Figure 1. This soil, with 86% passing No. 200 sieve, a Liquid Limit of 35.5% and a Plasticity Index of 19, is classified as A-6(11) by the American Association of State Highway Officials Classification[19],

(19) Standard Specifications for Highway Materials and Methods of Sampling and Testing, Part I, pp. 45-51, 1955.

and it is described as "a plastic clay soil usually having 75 percent or more passing the No. 200 sieve. Materials of this group have a high volume change between wet and dry states. This group index values range from 1 to 16, with increasing values indicating the combined effect of increasing plasticity indices and decreasing percentages of coarse material".

The PCA Soil Primer[20] describes group A-6 soils as "soils possessing

(20) Portland Cement Association, PCA Soil Primer, p. 41.

little internal friction and have low stability at the higher moisture contents. These soils are not suitable for use as subgrades under thin flexible base courses or bituminous surfaces because of large volume changes that are caused by moisture changes, and the loss of bearing power after the entrance of moisture. The heavier A-6 soils may require insulating courses to prevent excessive concrete pavement distortion or mud-pumping. All flexible-type bases must have an insulating course of A-1 or A-2 soils, stone chips, etc., or soil cement to prevent the clay

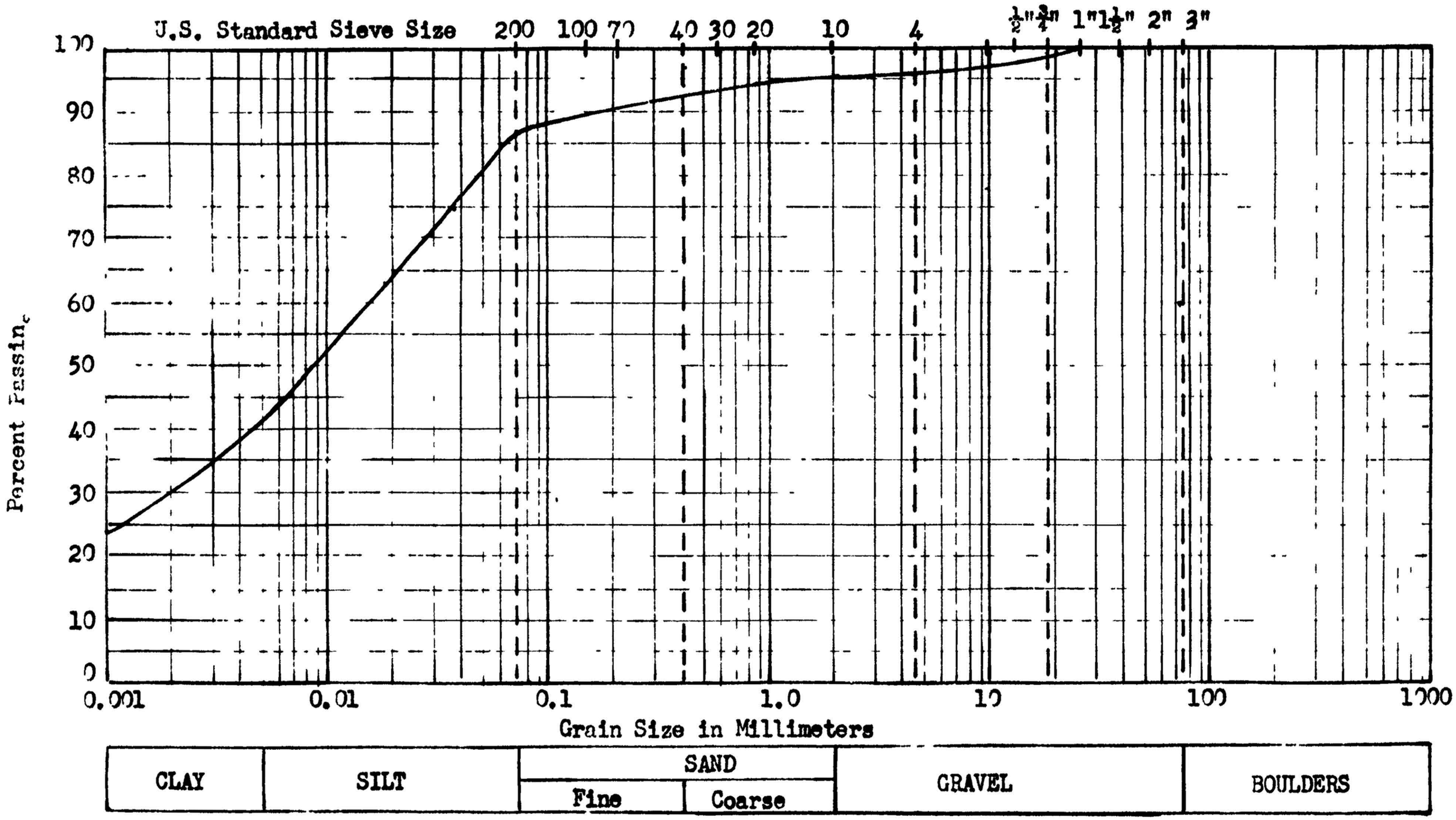


FIGURE 1

GRAIN SIZE ACCUMULATION CHART

from working into the flexible base, thus destroying its load-carrying capacity". This soil then is a very poor material for highway construction purposes.

All the soils were allowed to air-dry by spreading them on a wooden board for at least 7 days before they were sieved through the No. 4 sieve. Materials passing a No. 4 sieve were stored in metal containers and covered with a sheet of plastic. Hygroscopic moisture content of the soil in each container was determined, and the soil was corrected for hygroscopic moisture when used for preparation of specimens.

Hydrated Lime: The hydrated lime used in the experiments was manufactured by Ash Grove Lime and Cement Company at Kansas City, Missouri. It is ordinary commercial grade lime.

Cement: All the cement used in the experiments was of Type I portland cement, manufactured by Ash Grove Lime and Cement Company at Kansas City, Missouri.

EXPERIMENTS

In order to evaluate and compare the change in physical properties of the selected clay soil by the admixing of hydrated lime and cement, the following experiments were performed:

1. Liquid Limit Test
2. Plastic Limit Test
3. Shrinkage Test
4. Moisture-Density Relation Test
5. Unconfined Compression Test
6. Triaxial Compression Test
7. Penetration Test
8. Freeze-Thaw Test

The above experiments were conducted for the untreated soil, for soil-lime mixtures with 2, 4, 6, 8 and 10 percent of lime by weight, and for soil-cement with 2, 4, 6, 8 and 10 percent of cement by weight.

The liquid limit test, the plastic limit test and the shrinkage test are also called the Atterberg limit tests. These tests determine the plasticity characteristics of the total soil binder mixture. It can also be said that Atterberg limits reveal the probable behavior of the material in a roadway. The Atterberg limits of soil and soil mixtures were determined so as to establish their subgrade classification according to soil engineering procedure, and for the purpose of obtaining a measure of the improvement of their engineering characteristics caused by lime and cement treatment.

In order to compare the strengths of the various soil-lime and soil-cement mixtures, it was decided to conduct the strength tests on the mixtures compacted at the optimum moisture condition with a compactive effort approximating standard Proctor compaction. The moisture-density tests were therefore performed in order to find the optimum moisture content of each of the soil-lime and soil-cement mixtures. These tests also show any change in the maximum dry density with constant compactive effort, which would have a corresponding effect on the strength.

The unconfined compression test and the triaxial compression test were performed in order to measure the increase in strength of the soil with the addition of various percentages of lime and cement. From these tests we can determine the cohesion C and the angle of internal friction ϕ of the soil, which determine the shear strength of the soil, as given by Coulomb's empirical formula.

$$S = N \tan \phi + c$$

in which S = shearing strength, in lbs. per sq. ft.
C = cohesion, in lbs. per sq. ft.
N = pressure normal to shear plane in lbs. per sq. ft.
ϕ = angle of friction
$\tan \phi$ = coefficient of friction

In the case of a clay soil the purpose of stabilization is to increase the shear strength by increasing c and ϕ, particularly ϕ.

The penetration test is an empirical comparative bearing capacity test, and was performed to note the increase in bearing capacity (which is directly related to the shear strength) with the addition of various percentages of lime and cement.

The freeze-thaw test was conducted in order to determine whether the additives have any effect on the resistance to deterioration due to freezing and thawing cycles.

PROCEDURE

Air-dry soil passing No. 40 sieve was used in the Atterberg limit tests. In each case various percentages of lime and cement were mixed dry with the soil until the color of the mixtures appeared to be uniform. The correct amount of water was then added and thoroughly mixed with the soil.

Liquid Limit Tests

Liquid limit tests for the various percentages of lime and cement were performed in accordance with ASTM Tentative Method of Test, Designation: D423-54[21]. All the values and the graphs plotted from these

(21) ASTM Standards, op. cit., pp. 1269-1773.

values, are shown in Appendix A.

Plastic Limit Tests

Plastic limit tests for different mixtures were performed in accordance with ASTM Tentative Method, Designation: D424-54T[22].

(22) ASTM Standard, op. cit., pp. 1774-1776.

The results of the above two tests and the respective plasticity indices are shown in Table 1.

Shrinkage Limit Tests

Shrinkage limit tests for the various mixtures of soil-lime and soil-cement were conducted in accordance with ASTM Standard Method, Designation: D427-39[23].

(23) ASTM Standard, op. cit., pp. 1782-1785.

The values of shrinkage limit, shrinkage ratio and volumetic change for the respective mixtures are shown in Table 2.

MIXTURE	LIQUID LIMIT PERCENT	PLASTIC LIMIT PERCENT	PLASTICITY INDEX
Natural Soil	35.5	19.0	16.5
Soil + 2% Lime	34.2	27.9	6.3
Soil + 4% Lime	33.7	29.2	4.5
Soil + 6% Lime	33.1	30.1	3.0
Soil + 8% Lime	31.4	——	
Soil + 10% Lime	30.2	——	
Soil + 2% Cement	33.6	18.9	14.7
Soil + 4% Cement	33.2	21.2	12.0
Soil + 6% Cement	33.9	22.6	11.3
Soil + 8% Cement	32.2	21.6	10.6
Soil + 10% Cement	32.5	23.0	9.5

Table 1

RESULTS OF PLASTICITY TESTS ON SOIL AND VARIOUS MIXTURES OF SOIL-LIME AND SOIL-CEMENT

MIXTURE	SHRINKAGE LIMIT PERCENT	SHRINKAGE RATIO	VOLUMETRIC CHANGE PERCENT
Natural Soil	17.88	1.74	21.09
Soil + 2% Lime	19.12	1.73	18.82
Soil + 4% Lime	24.12	1.52	8.94
Soil + 6% Lime	26.19	1.49	5.68
Soil + 8% Lime	26.48	1.48	5.21
Soil + 10% Lime	25.89	1.47	6.04
Soil + 2% Cement	18.00	1.74	20.88
Soil + 4% Cement	18.93	1.72	19.04
Soil + 6% Cement	18.32	1.71	19.97
Soil + 8% Cement	18.88	1.72	19.13
Soil + 10% Cement	19.53	1.69	17.69

Table 2

Results of Shrinkage Tests on Soil and Various Mixtures of Soil-Lime and Soil-Cement.

Moisture-Density Relations Tests

Air-dry soil passing a No. 4 sieve was used for moisture-density relation tests. The soil was mixed dry with each percentage of admixture. Water was added, and by means of a trowel, the mixture was mixed for three minutes. It was then mixed by a mechanical mixer for another three minutes. The mechanical mixer used was a Lancaster Counter Current Mixer, manufactured by Posey Iron Works, Inc., Lancaster, Pennsylvania. See Figure 2.

The mold used for compaction was an aluminum split mold made by Soiltest, Inc., Chicago, Illinois. It is 7 1/2 in. high and 2.8 in. inside diameter. The volume of the mold is 0.0267 cu. ft. The compactor used was an aluminum bar 18 1/2 in. long, 1 1/2 in. diameter and weighing 3.32 lbs. This bar was fitted in a sleeve made of galvanized sheet iron which acted as a guide. A string was attached to the aluminum bar and the sleeve so that the drop of the bar was controlled at one foot. Figure 3 shows the compactor and the mold.

The mixture was compacted in the mold in five layers, twenty blows per layer, thus providing a compactive effort of 7.18 ft. lbs. per cu. in. as compared to the compactive effort of 7.13 ft. lbs. per cu. in. for Standard Proctor Compaction.

The procedure used for finding the optimum moisture content of various mixtures of soil-lime and soil-cement was in accordance with the Tentative Method of Test for Moisture-Density Relation of Soils, ASTM Designation: D698-42T[24]. The results of these tests and the

(24) ASTM Standard, op. cit., pp. 1789-1791.

graphs drawn from the values determined are shown in Appendix B. The optimum moisture content and maximum dry density for various admixtures are shown in Table 3.

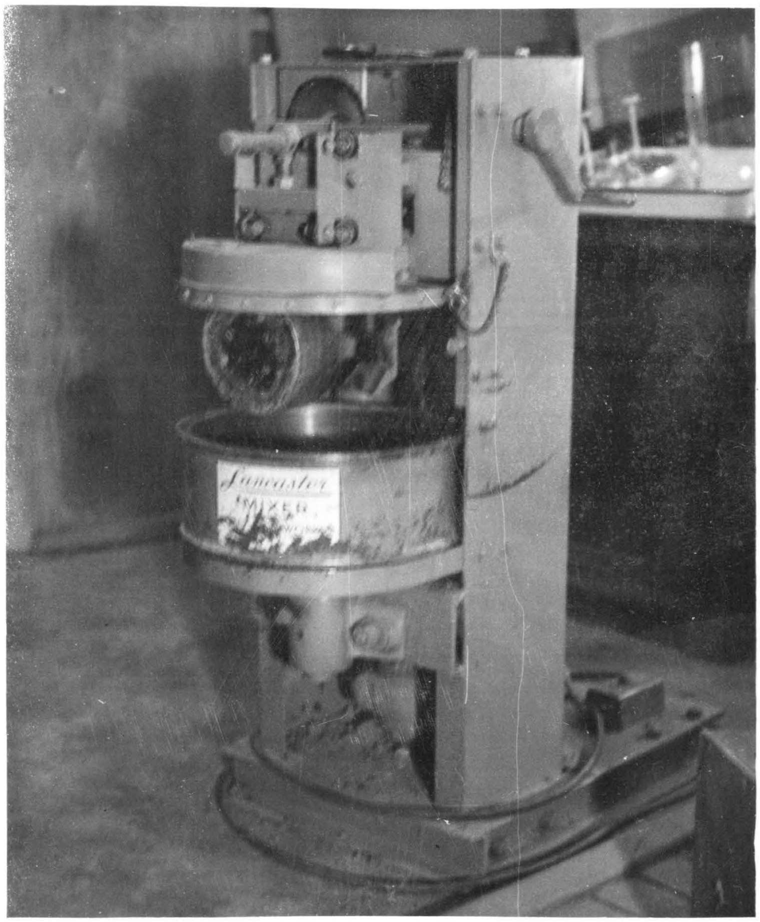


FIGURE 2

LANCASTER COUNTER BATCH MIXER

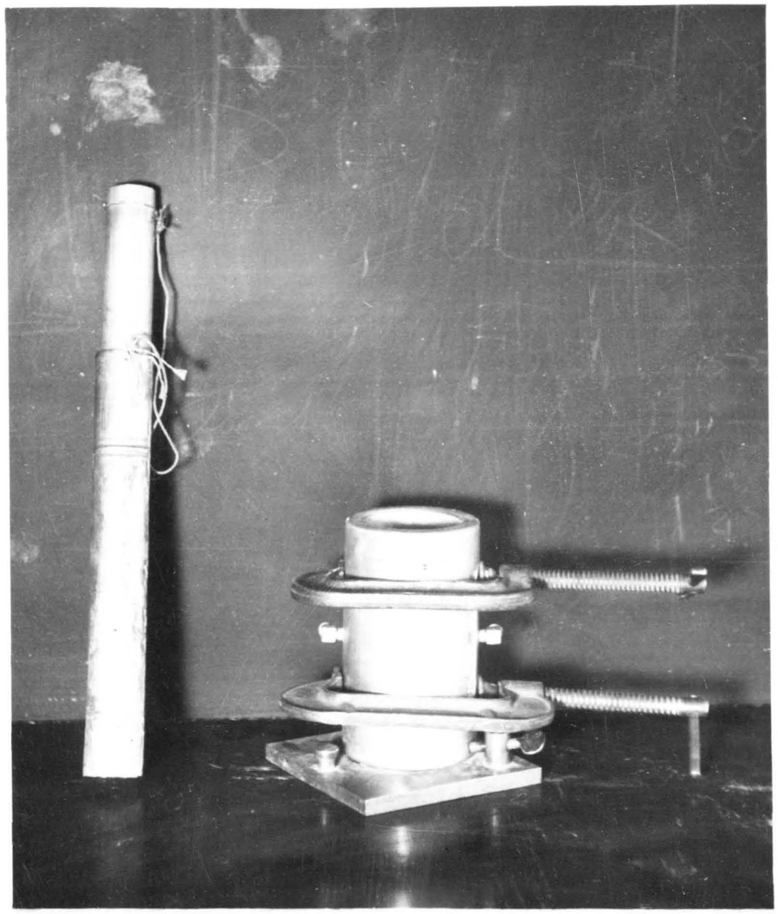


FIGURE 3

SPLIT MOULD AND COMPACTOR

MIXTURE	MAXIMUM DRY DENSITY LBS. PER CU. FT.	OPTIMUM MOISTURE CONTENT PERCENT
Natural Soil	102.5	19.8
Soil + 2% Lime	99.3	21.8
Soil + 4% Lime	97.6	22.0
Soil + 6% Lime	97.2	22.4
Soil + 8% Lime	97.0	23.5
Soil + 10% Lime	94.3	24.1
Soil + 2% Cement	100.7	21.2
Soil + 4% Cement	100.4	20.5
Soil + 6% Cement	102.4	20.6
Soil + 8% Cement	101.8	19.7
Soil + 10% Cement	103.9	20.8

Table 3

Results of Moisture-Density Relations Tests on Soil and Various Mixtures of Soil-Lime and Soil-Cement.

Specimens for unconfined compression tests, triaxial compression tests, penetration tests and freeze-thaw tests, were compacted at their respective optimum moisture contents, which were determined from the above moisture-density relation tests. These specimens were compacted in the manner described above.

Unconfined Compression Tests

For the unconfined compression tests, four specimens were prepared for each percentage of both admixtures. Two of these specimens were tested after 7 days of moist curing, the remaining two were tested after 28 days. The specimens were extruded from the mold and trimmed to a height of 6 in. in order to fit the triaxial testing machine, the description of which is given below. No lateral pressures were applied to the specimens in this test. The specimens were loaded at a constant rate of 0.125 in. per minute until failure occurred.

The results of the unconfined compression tests are presented in Table 4. Typical failure of the specimens are shown in Figure 4.

Confined Compression Tests

Six specimens for each percentage of both admixtures were prepared as described above. After 7 days of moist curing, they were tested in the triaxial testing machine. Three different lateral pressures, namely, 5, 10 and 15 psi were used. Two of the 6 specimens were tested with each of the lateral pressures indicated above.

The triaxial apparatus used was Model T-115-1 manufactured by Soiltest, Inc. of Chicago, Illinois. See Figure 5. It consists of a 8 x 8 x 3/4 in. base plate mounted on the desk of a frame, a [illegible] chamber head fitted with a sleeve through which a load piston may be passed, and a loading mechanism. The loading mechanism consists of a 1/8 horse power motor connected to a threaded vertical shaft through a

MIXTURE	7 Days ULTIMATE STRESS P.S.I.	28 Days ULTIMATE STRESS P.S.I.	INCREASE IN STRESS PERCENT
Natural Soil	46.1	48.2	----
Soil + 2% Lime	63.0	85.3	35.4
Soil + 4% Lime	75.5	104.7	38.7
Soil + 6% Lime	92.2	124.7	35.2
Soil + 8% Lime	99.4	126.7	27.5
Soil + 10% Lime	101.5	131.2	29.3
Soil + 2% Cement	58.5	95.2	62.7
Soil + 4% Cement	69.0	107.0	55.1
Soil + 6% Cement	99.1	148.5	49.8
Soil + 8% Cement	109.5	155.0	41.6
Soil + 10% Cement	106.4	157.4	47.9

Table 4

Results of Unconfined Compression Tests on Soil and Various Mixtures of Soil-Lime and Soil-Cement at 7 days and 28 days.

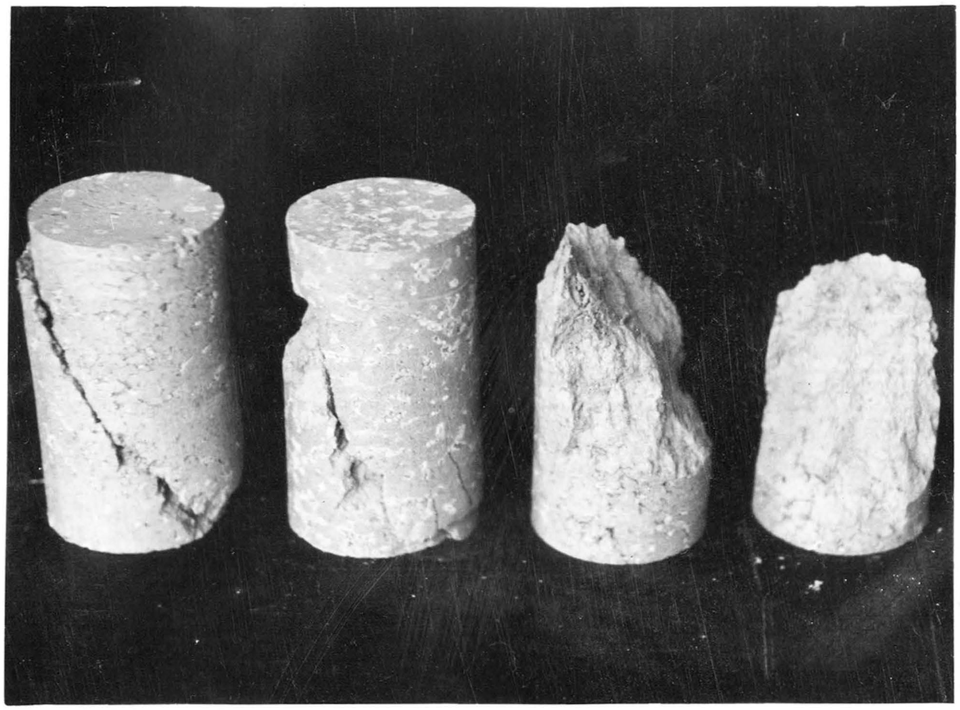


FIGURE 4

TYPICAL FAILURES OF UNCONFINED COMPRESSION TEST SPECIMENS

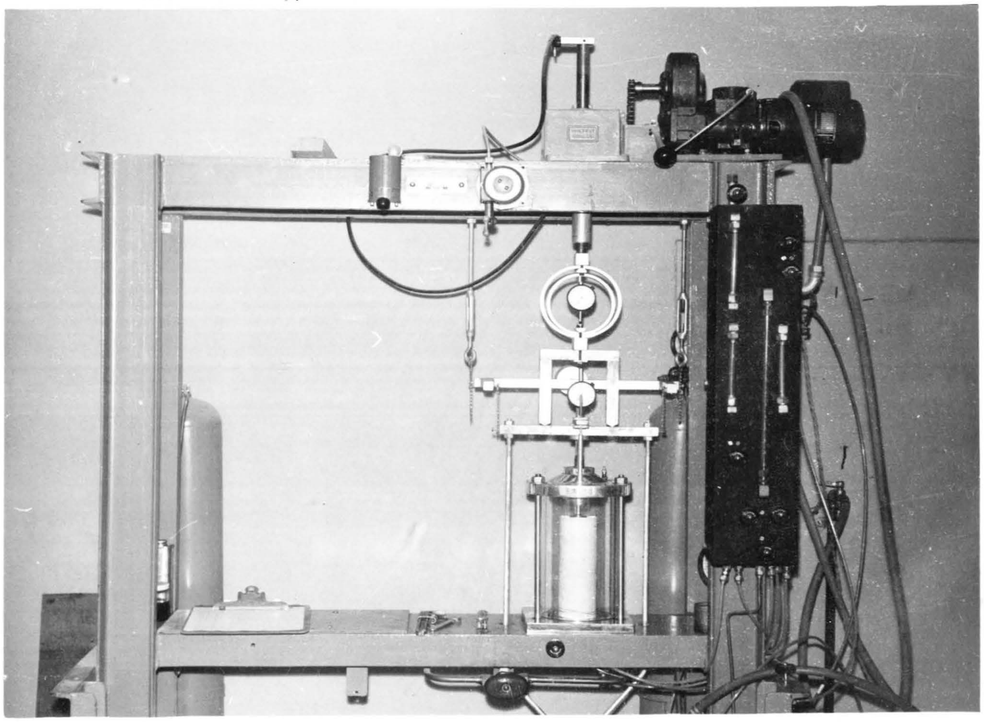


FIGURE 5

TRIAXIAL TESTING MACHINE

variable speed transmission. A 1500 lbs. capacity double proving ring is attached at the end of the vertical shaft.

The specimen to be tested was placed on the pedestal, and was enclosed in a rubber sleeve. Glycerine was introduced into the lucite chamber, and the required pressure was applied to the glycerine by means of compressed air. See Figure 6. The specimen was then loaded by driving the vertical shaft down at a constant rate of 0.125 in. per minute until the specimen failed, as indicated by the progressive falling off of load.

The results of the triaxial tests and the Mohr's circles of stress constructed from the values obtained are presented in Appendix C. The values of the angle of internal friction and cohesion obtained from each set of Mohr's circles are presented in Table 5. The values of angle of internal friction and cohesion plotted against their respective percentage of admixture are shown in Figure 7. Typical failures of the specimens are shown in Figure 8.

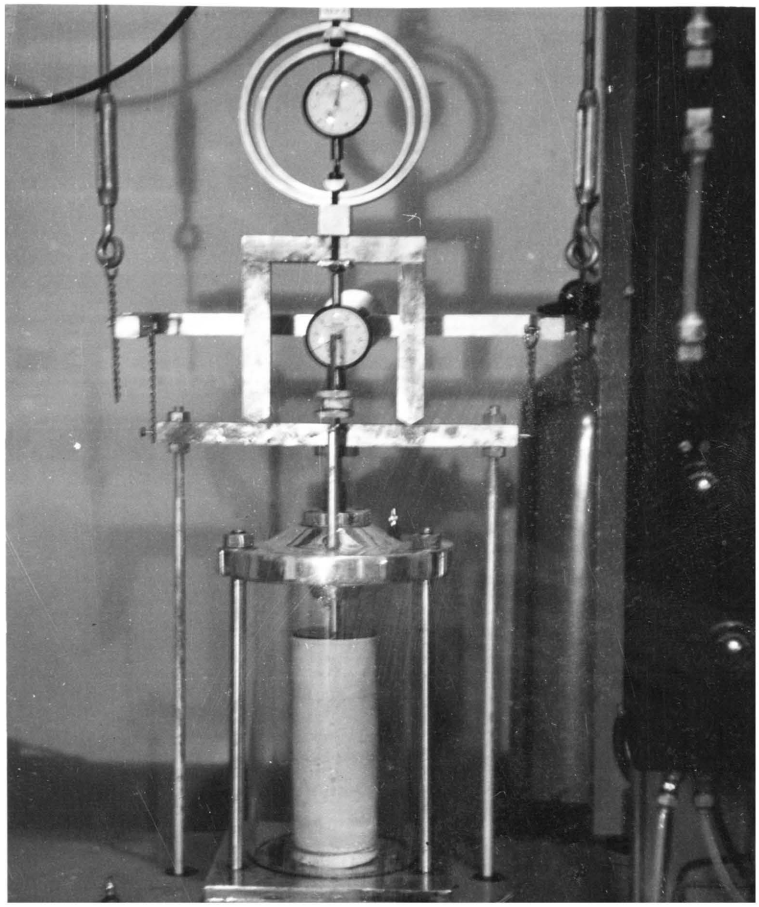


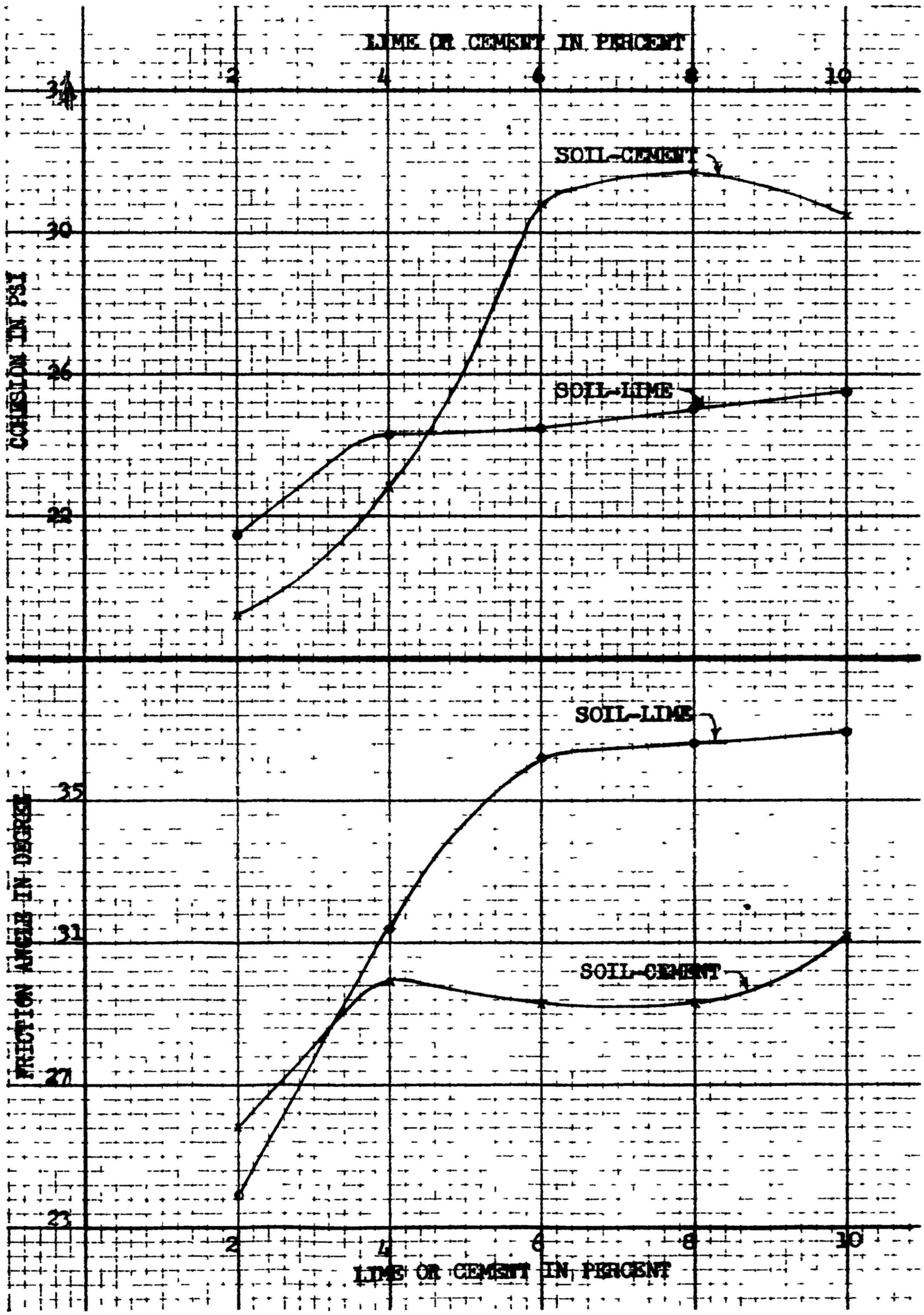
FIGURE 6

INSTRUMENTATION OF TRIAXIAL COMPRESSION TEST

MIXTURE	COHESION C P.S.I.	FRICTION ANGLE ϕ DEGREE
Natural Soil	17.0	16.6
Soil + 2% Lime	21.5	23.9
Soil + 4% Lime	24.3	31.4
Soil + 6% Lime	24.5	36.2
Soil + 8% Lime	25.0	36.6
Soil + 10% Lime	25.5	36.9
Soil + 2% Cement	19.2	25.8
Soil + 4% Cement	22.8	29.9
Soil + 6% Cement	30.8	29.3
Soil + 8% Cement	31.7	29.3
Soil + 10% Cement	30.5	31.2

Table 5

Results of Confined Compression Tests on Soil and Various Various Mixtures of Soil-Lime and Soil-Cement.



LIME OR CEMENT IN PERCENT
2
4
6
8
10
COHESION IN PSI
30
26
22
SOIL-CEMENT
SOIL-LIME
FRICTION ANGLE IN DEGREE
35
31
27
23
SOIL-LIME
SOIL-CEMENT
LIME OR CEMENT IN PERCENT

FIGURE 7

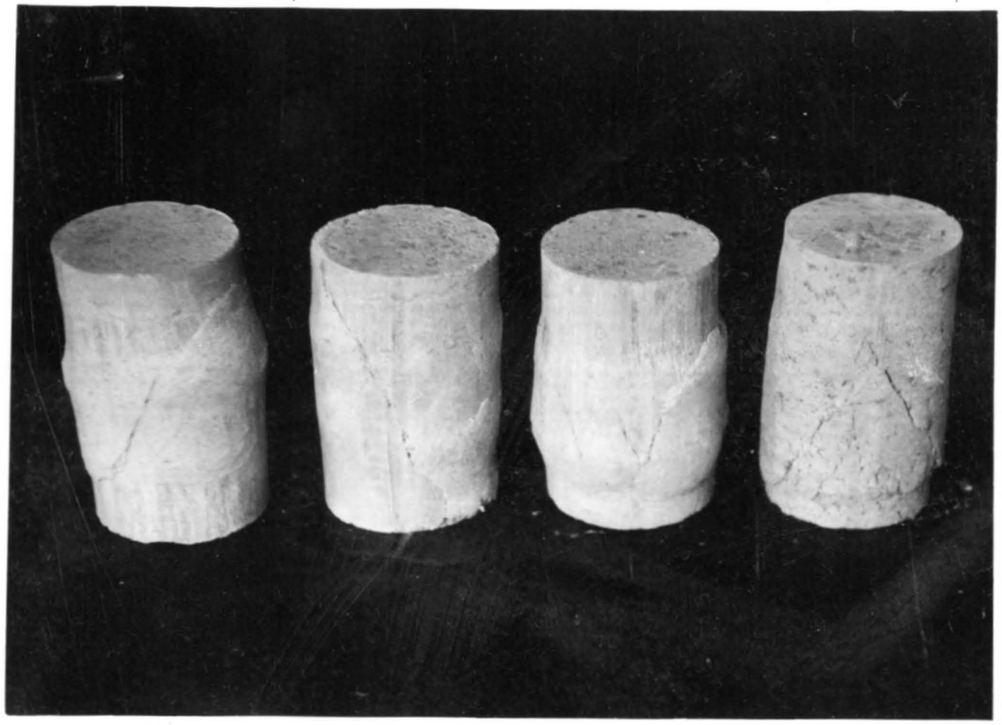


FIGURE 8

TYPICAL FAILURES OF TRIAXIAL COMPRESSION TEST SPECIMENS

Penetration Tests

For penetration tests, two specimens for each percentage of both admixtures were prepared as described previously. These specimens were not trimmed; they were 7 1/2 in. high. After 7 days of moist curing they were enclosed in the split mold, and placed on the machine for testing as shown in Figure 9. The piston having an area of 1/2 sq. in. was used, and the rate of penetration was 1/2 in. per minute. The load required for the penetration of one inch was recorded in each case. The values shown in Table 6 are the average values of two specimens tested.

Freeze-Thaw Tests

One specimen for each percentage of both admixtures was prepared as described previously. These specimens were trimmed to 4 in. high so that they could be placed in the freezing compartment of the refrigerator. The samples were first moist cured for seven days, after which they were subjected to successive freezing and thawing (24 hours freezing and 24 hours thawing) for 12 days. Figures 10 & 11 show the specimens after 6 cycles of the freeze-thaw test.

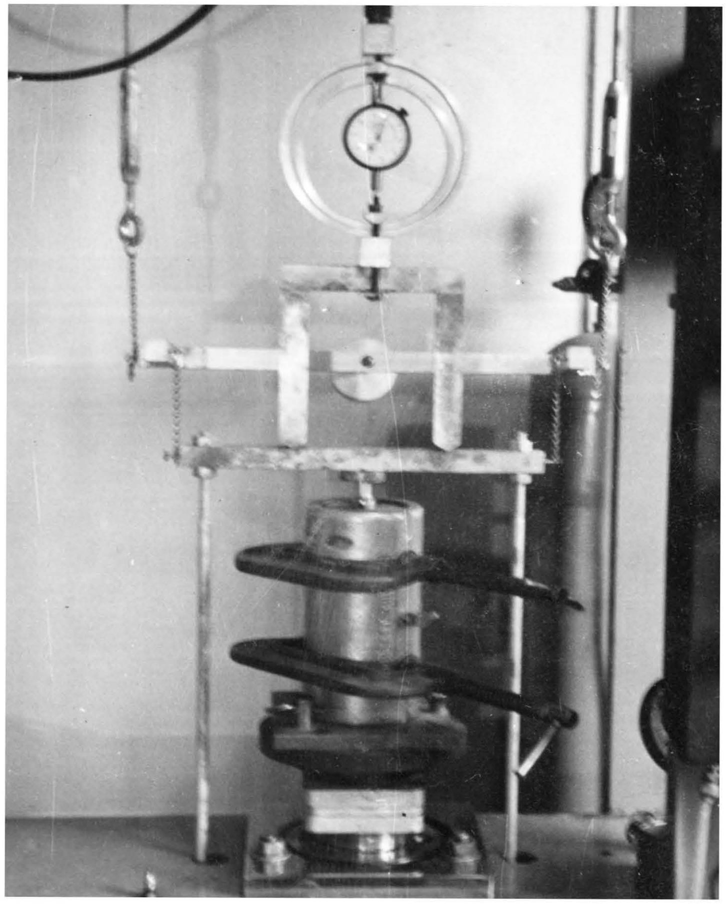


FIGURE 9

SPECIMEN ENCLOSED IN THE MOULD FOR PENETRATION TEST. PISTON HAVING 1/2 SQ. IN. AREA IS SHOWN AT THE CENTER ON TOP OF SPECIMEN.

MIXTURE	*LOAD IN LBS.	**RATIO
Natural Soil	234	1.00
Soil + 2% Lime	267	1.14
Soil + 4% Lime	487	2.08
Soil + 6% Lime	544	2.32
Soil + 8% Lime	631	2.70
Soil + 10% Lime	560	2.40
Soil + 2% Cement	242	1.03
Soil + 4% Cement	275	1.17
Soil + 6% Cement	297	1.27
Soil + 8% Cement	363	1.55
Soil + 10% Cement	526	2.25

Table 6

Results of Penetration Tests on Soil and Various Mixtures of Soil-Lime and Soil-Cement.

*Load required for 1 in. penetration of a piston having 1/2 sq. in. area, at the rate of 1/2 in. per minute.

**Ratio of the penetration resistance of the mixture to that of the natural soil.

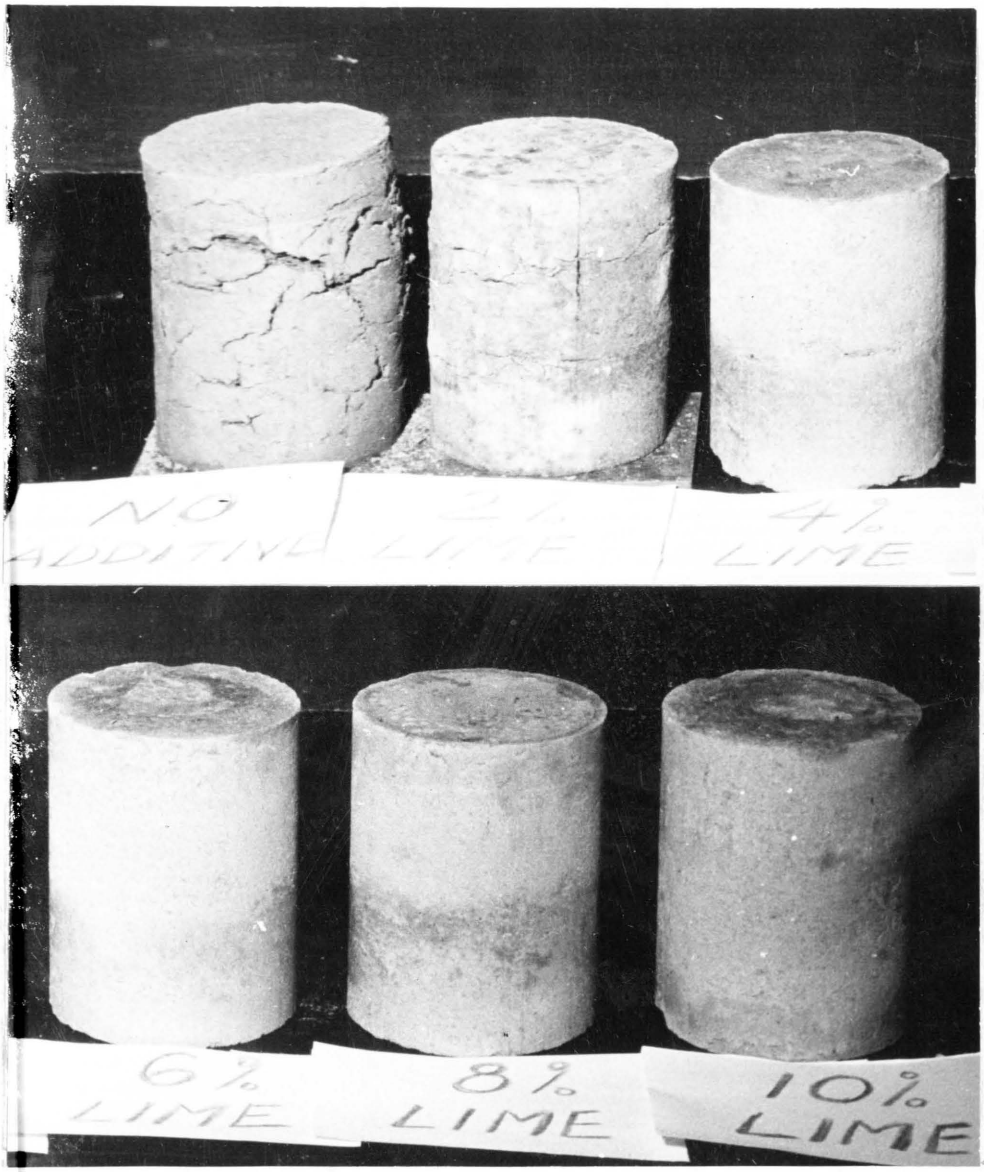


FIGURE 10

SPECIMENS OF SOIL AND VARIOUS MIXTURES OF SOIL-LIME AFTER 6 CYCLES OF FREEZE-THAW TESTS.

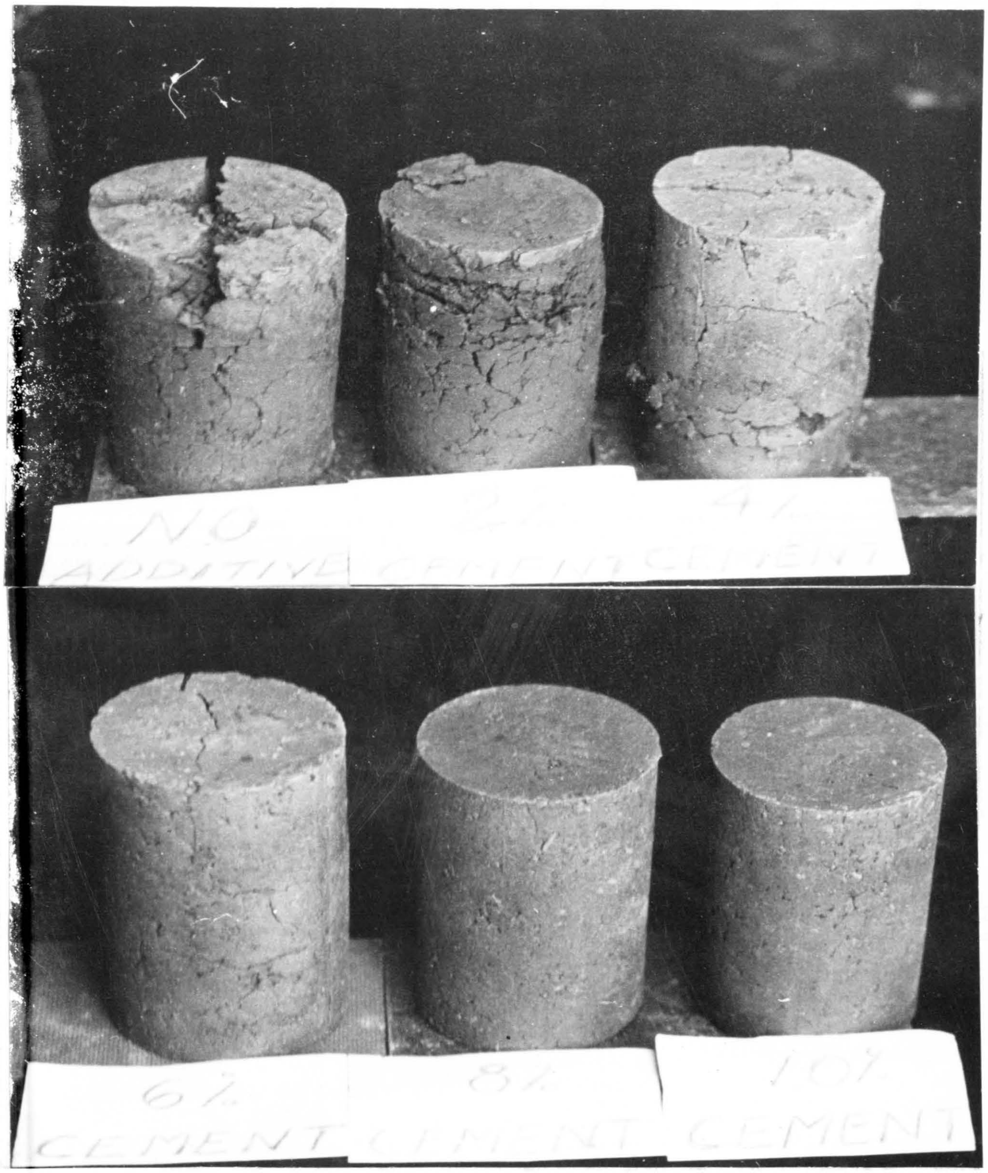


FIGURE 11

SPECIMENS OF SOIL AND VARIOUS MIXTURES OF SOIL-CEMENT AFTER 6 CYCLES OF FREEZE-THAW TESTS.

DISCUSSION OF RESULTS

The process of stabilization of subgrade using hydrated lime or cement usually involves the following steps: (1) scarifying the roadbed; (2) pulverizing the soil; (3) dry-mixing the soil and stabilizer; (4) adding sufficient water and mixing, and (5) compaction. For a clay soil, it is difficult to carry out the wet mixing and compaction operation satisfactorily because of the plasticity or stickiness of the soil. In such cases, if the clay soil can be made less plastic, construction operations may be conducted more efficiently.

During the moisture-density relation test, it was observed that lumps of clay were formed when the untreated soil was mixed in a mechanical mixer. When the soil was mixed with 2 percent lime, there were considerably fewer clay lumps. With the addition of 4 percent or more lime, a drastic change in texture was noted. The material became friable and had the characteristics of a non-plastic mix. Figure 12 shows the texture of various percentages of lime-soil mixtures. In the case of cement admixture, no such change was observed. The texture of the soil was not modified even with the addition of 10 percent cement. See Figure 13.

The results of plasticity tests on soil and various mixtures of soil-lime and soil-cement are presented in Table 1. It will be seen that lime and cement admixtures had very little effect on the liquid limit of the soil. With the addition of only 2 percent lime, the plastic limit was increased from 19.0 percent to 27.9 percent, thereby the plasticity index was reduced considerably. But the plastic limit of the soil remained essentially the same when 2 percent cement was added. It was not possible to obtain the plastic limit of the mixture containing 8 percent or more lime, since the mixture crumbled when rolled with the palm. The

FIGURE 12

TEXTURE OF SOIL AND VARIOUS MIXTURES OF SOIL-LIME AFTER MIXING WITH A MECHANICAL MIXER.

FIGURE 13

TEXTURE OF SOIL AND VARIOUS MIXTURES OF SOIL-CEMENT AFTER MIXING WITH A MECHANICAL MIXER.

results clearly shows that lime admixture is much more effective than cement in reducing the plasticity index of this soil.

It is inevitable that the moisture content of the highway subgrade will vary somewhat with climatic changes. A soil is considered undesirable for subgrade material if it undergoes a large volume change with a small change in moisture content. The results of the shrinkage tests in Table 2 indicate definite improvement in shrinkage properties of the soil by the use of admixtures. It will be observed that lime is more effective than cement in improving the shrinkage properties of the soil. By adding 2 percent lime the volumetric change was reduced from 21.09 percent to 18.82 percent. To accomplish the same improvement, it would be necessary to use about 9 percent cement.

The results of compaction tests presented in Table 3 show that the optimum moisture content remains essentially the same with the addition of up to 10 percent cement. The optimum moisture content was increased slightly by lime addition. The addition of either lime or cement did not improve the density characteristics. In fact they appear to be affected somewhat adversely. The addition of 4, 6 and 8 percent of lime to the soil decreased the dry density of the soil from 102.5 lbs. per cu. ft. to about 99.3 lbs. per cu. ft. This adverse effect cannot be considered significant in view of the very favorable modification of the soil with respect to plasticity and shrinkage properties.

When a soil cylinder is loaded vertically, it fails in shear. Before failure, it may be considered that cones are formed at each end of the cylinder. As vertical deformation proceeds, these cones move closer together, causing the soil to bulge and ultimately fail by splitting off the sides of the cylinder. The cylinder might fail along any planes which are between the cones and parallel to the cone surfaces.

The angle, α , made by surfaces of the cone with the horizontal, is theoretically equal to 45 degree + $\phi/2$, ϕ being the angle of internal friction along the surfaces of the cones at the instant the cylinder fails. See Figure 14.

Theoretically, the ultimate vertical pressure $V = 2C \tan\alpha$, in which V = vertical pressure in lbs. per sq. ft.; C = cohesion in lbs. per sq. ft. and α = the angle made by surfaces of the cone with the horizontal, in degrees.

The data could be analyzed by means of Mohr's circle as follows:

With a center at a distance $V/2$ from the origin, an arc of circle with radius $V/2$ is constructed as shown in Figure 15, and a line drawn through the origin at an angle α with the horizon, until it intersects the arc at some point M. The vertical projection of OM then gives the shear stress graphically, according to Mohr; and the horizontal projection of OM gives the normal stress along the surfaces of the cones at the instant the cylinder fails.

A straight line drawn tangent to the arc at the intersection point M shows the relation of shear stress S to normal stress N. The intercept of this line on the vertical axis gives cohesion C, and the angle this line makes with the horizontal gives the angle of internal friction ϕ.

However, the angle of fracture α , is not always defined clearly by the tested sample. Therefore, in the unconfined compression test, values of C and ϕ cannot be determined.

In the case of triaxial compression test, different lateral pressures, G_1, G_2, etc. can be applied to the specimens, and corresponding stress circles can be constructed with a radius of $\frac{V - G}{2}$ and a center at $\frac{V + G}{2}$ distance from the origin. A line tangent to these circles can be drawn and the values of C and ϕ can be determined graphically as described above.

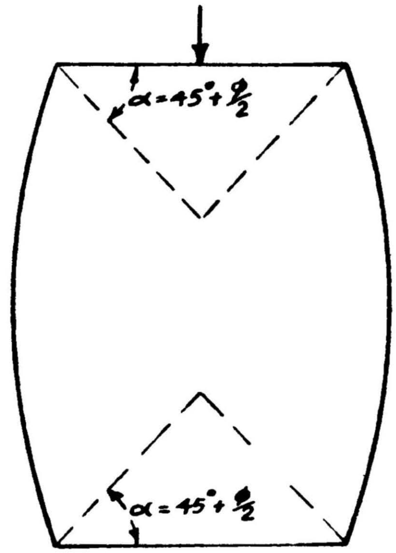


Figure 14

Theoretical Failure of Soil Cylinder when Vertically Loaded.

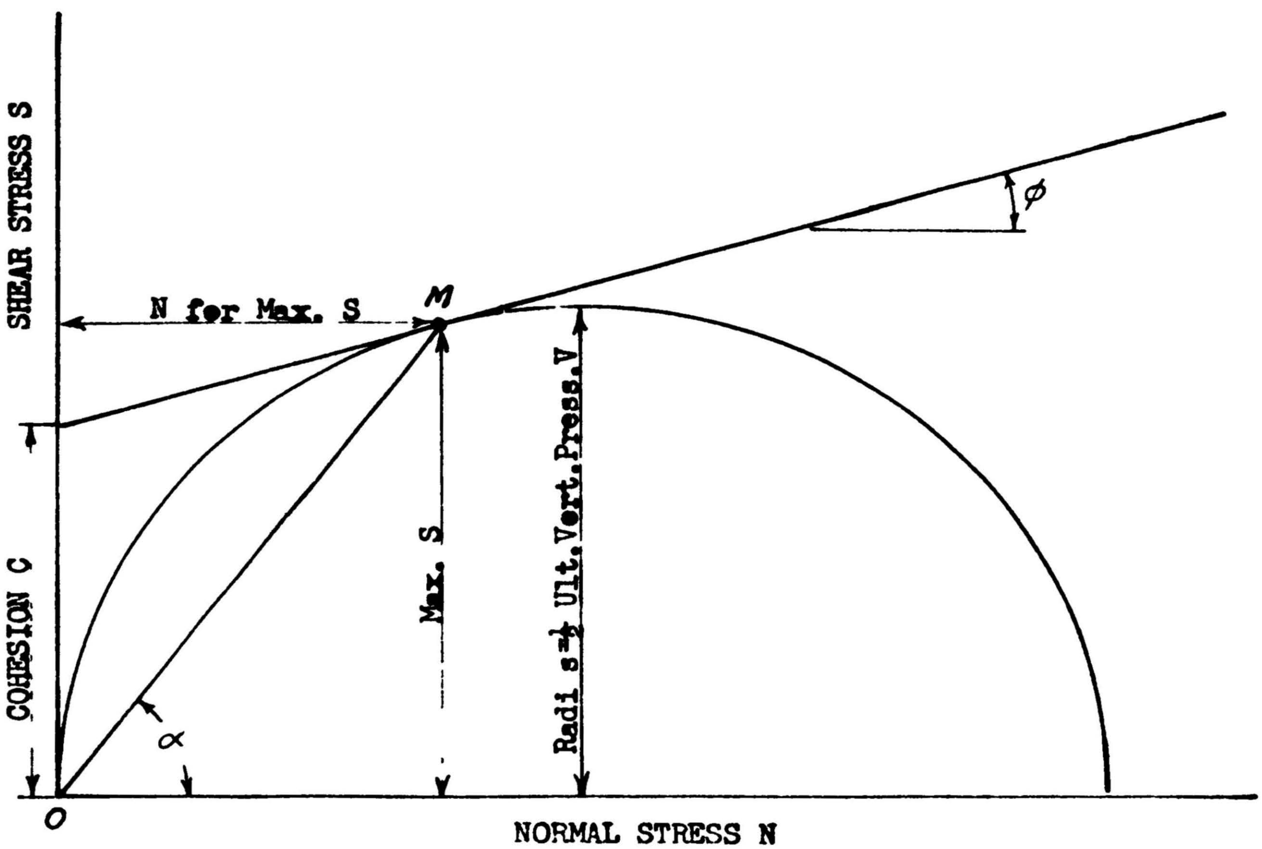


Figure 15

Graphical Representation of Stresses in Cylinder at Failure.

Since the results of the unconfined compression tests cannot be presented in terms of cohesion and angle of internal friction, they are presented in Table 4 in terms of ultimate stress. For 7 days ultimate stress, it will be seen that up to 4 percent, the addition of lime produced stronger specimen. But from 6 to 10 percent, cement admixture produced better results. The average increase in ultimate stress of the soil-cement specimens resulted from additional curing (from 7 days to 28 days) is 51.4 percent, whereas for the soil-lime specimens, the average increase is only 33.2 percent. Since hydrated lime is known to react slowly with pozzolanic substances, it cannot be concluded at this point which of the additives will give the soil better strength over a long period. In general, strength was increased with the increase in both admixtures.

The results of the confined compression tests are tabulated in Table 5. They are also presented in the form of graphs in Figure 7. There was a marked increase in angle of internal f iction with the addition of lime up to 6 percent. Thereafter, the increase in lime admixture only resulted in a small increase in angle of friction. Cohesion increased from 17.0 psi for untreated soil to 24.3 psi when 4 percent lime was added. From 4 percent to 10 percent lime admixture, cohesion increased only slightly. For cement admixture, angle of internal friction increased rapidly up to 4 percent and the addition of 6 percent cement resulted in a pronounced increase in cohesion of the soil. Since the shear strength of the soil due to friction is more dependable than that due to cohesion, an increase in the angle of friction is of more value than the increase in cohesion. Therefore, lime seems to have a better effect on the strength of this soil than cement. For lime admixture, the addition of 6 percent will produce the best result, and for cement admixtures, the addition of from 4 to 6 percent will produce the best results.

Since the resistance to penetration of a piston is a measure of the strength of the specimen, the results of penetration tests shown in Table 6 indicate that the addition of both lime and cement increases the strength of the soil. It will be seen from the table that soil-lime specimens have a greater resistance to penetration than soil-cement specimens.

Photographs of specimens taken after six 48-hour cycles of freezing and thawing are shown in Figures 10 and 11. As can be observed, the specimen with 4 percent lime showed much less deterioration than the untreated soil specimen. If touched, the untreated soil sample would crumble. The specimen with 2 percent lime cannot be picked up by hand without separation at the compaction joint. The specimen with 4 percent lime still showed a slight crack at the compaction joint, but the specimens with 6 percent or more lime were in good condition. Using cement as admixture, even the specimen with 6 percent additive showed large cracks. Specimens with 8 and 10 percent cement were in good condition. It must be borne in mind that these specimens were subjected to freeze-thaw test after 7 days moist curing. There is no doubt that the length of curing has a favorable effect on the strength of specimens containing either admixture. It is apparent that if the specimens are cured over a longer period before subjecting them to freeze-thaw test, they will develop better resistance to freezing and thawing. Therefore, it is desirable to construct lime or cement stabilized bases early in the spring, so that additional strength may be developed before freezing and thawing occurs.

SUMMARY AND CONCLUSIONS

The data developed in this research has been discussed and analyzed in the light of the stated purpose of this investigation. Each of the tests conducted during this research reveal certain physical properties of the material. To place a value on each individual physical property would be unrealistic and impractical since each change is inter-related with another. The comments which follow summarize the general conclusion derived from the results of the various tests.

(1) Lime admixture is much more effective in reducing the plasticity of the soil than cement admixture.

(2) The use of either admixture improves the shrinkage properties of the soil but lime admixture gives a better result for the same percentage of added materials.

(3) Cement admixtures does not affect the moisture-density relations of the soil. The use of lime admixture increases the optimum moisture content slightly, and decreases the maximum dry density somewhat.

(4) In general, the use of either admixture increases the strength characteristics of the soil. Based on the unconfined compression test results, the increase in strength is proportional to the amount of admixtures.

(5) A greater increase in angle of internal friction can be obtained through the use of lime than through the use of cement. On the other hand, the use of cement admixture resulted in a greater increase in cohesion than did the use of lime. Since the increase in the angle of friction of a clay soil is more important than the increase in cohesion, then in general, the lime admixture gives a better improvement in the strength of the soil.

(6) For the same percentage of additive used, soil-cement specimens suffer more deterioration from freezing and thawing than soil-lime specimens.

This laboratory investigation indicates that the soil can be modified favorably by lime or cement admixture. Also, indications are that lime is more effective than cement and produces more extensive modification for the same amount of admixture.

Different soils possess different chemical and physical properties. Therefore, for any particular soil encountered, some laboratory investigation of the soil must be conducted before any field work is attempted.

The writer realizes that the favorable results achieved in this laboratory study probably could not be completely obtained in the field. However, further study of the different field operations involved in soil stabilization, particularly the mixing operation, and the observation of field results would appear to be justified.

Difficulty in mixing the soil and cement is often encountered in cement stabilization of clay soils. The possibility of overcoming this difficulty by adding cement in a form of "slurry" instead of "dry" was tried without much success by Mills.[25] In this respect, lime-cement stabilization has been tried with some success. The addition of lime reduces the plasticity of the clay, thereby permitting the cement to mix readily with the soil particles. The extra cost of using both admixtures may be offset by the fact that less manipulation will be required. Lime-cement stabilization will be adaptable to areas with poor drainage conditions and where a road to carry heavy traffic is needed. The use of lime alone may not provide the necessary strength. Besides, under such conditions, water may leach away the lime.

(25) Mills, H. W., op. cit., p. 326.

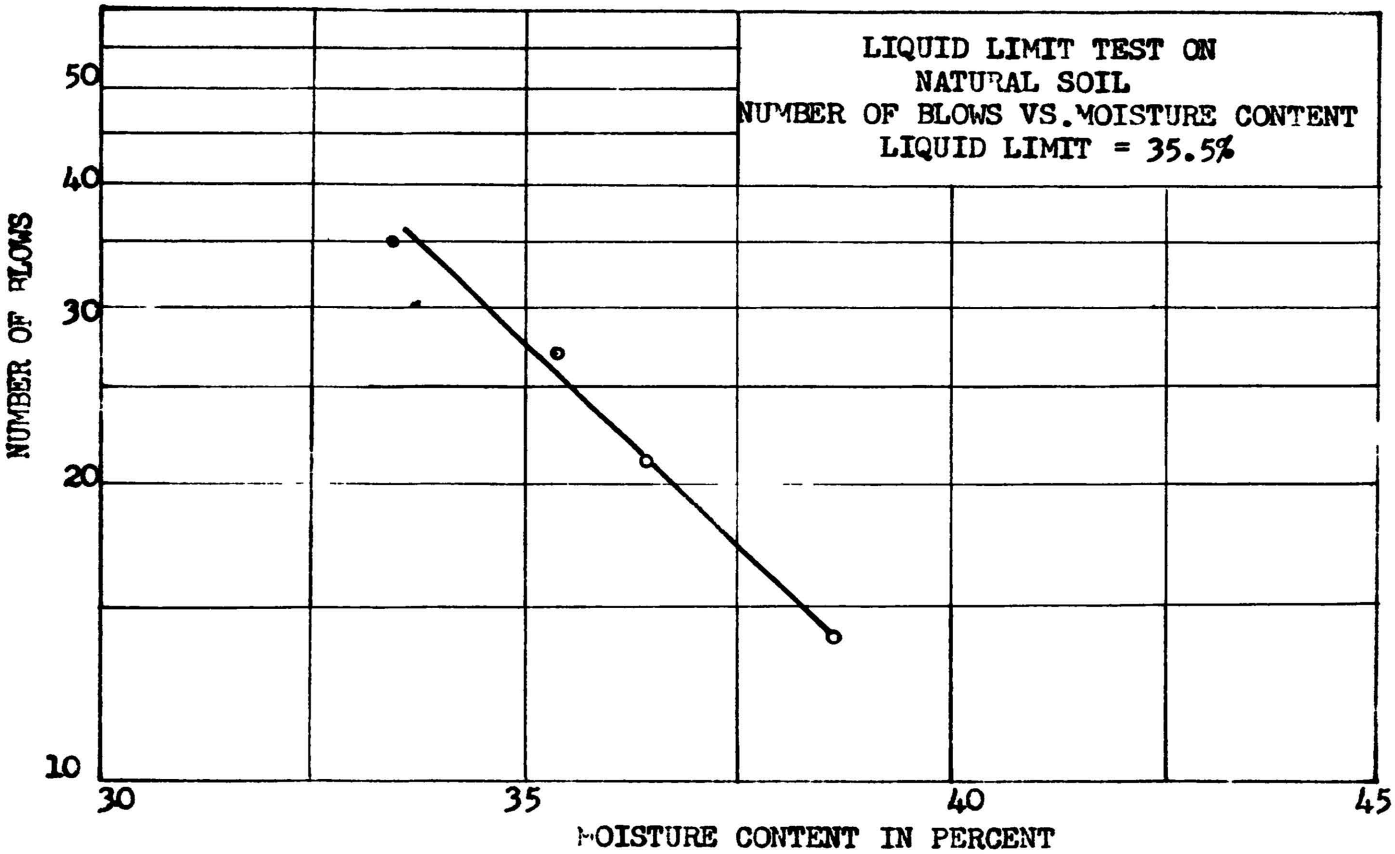
There is no published information concerning the laboratory experiments on lime-cement stabilization of soil, and some laboratory investigations of this type of stabilization would be justified.

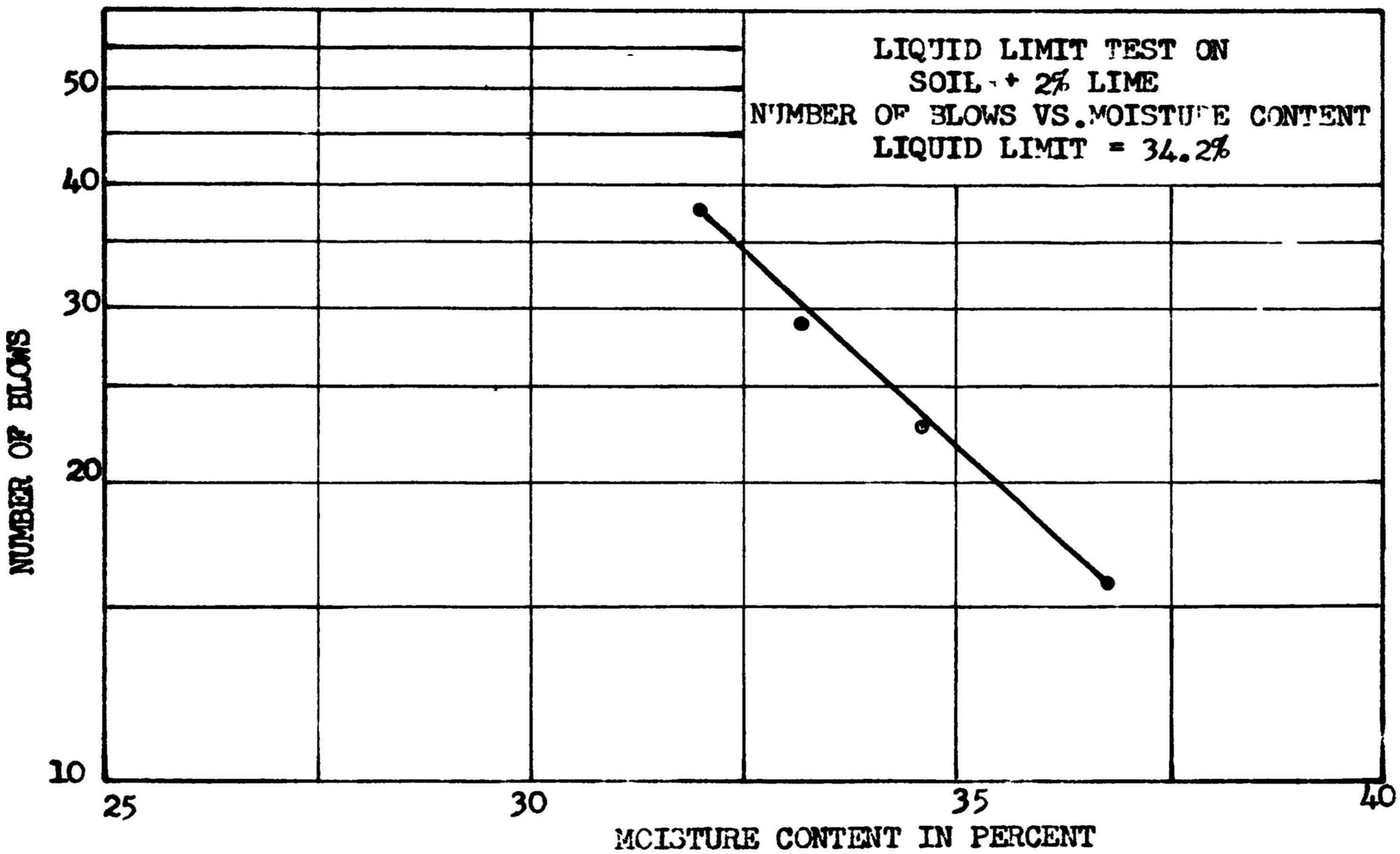
APPENDIX A

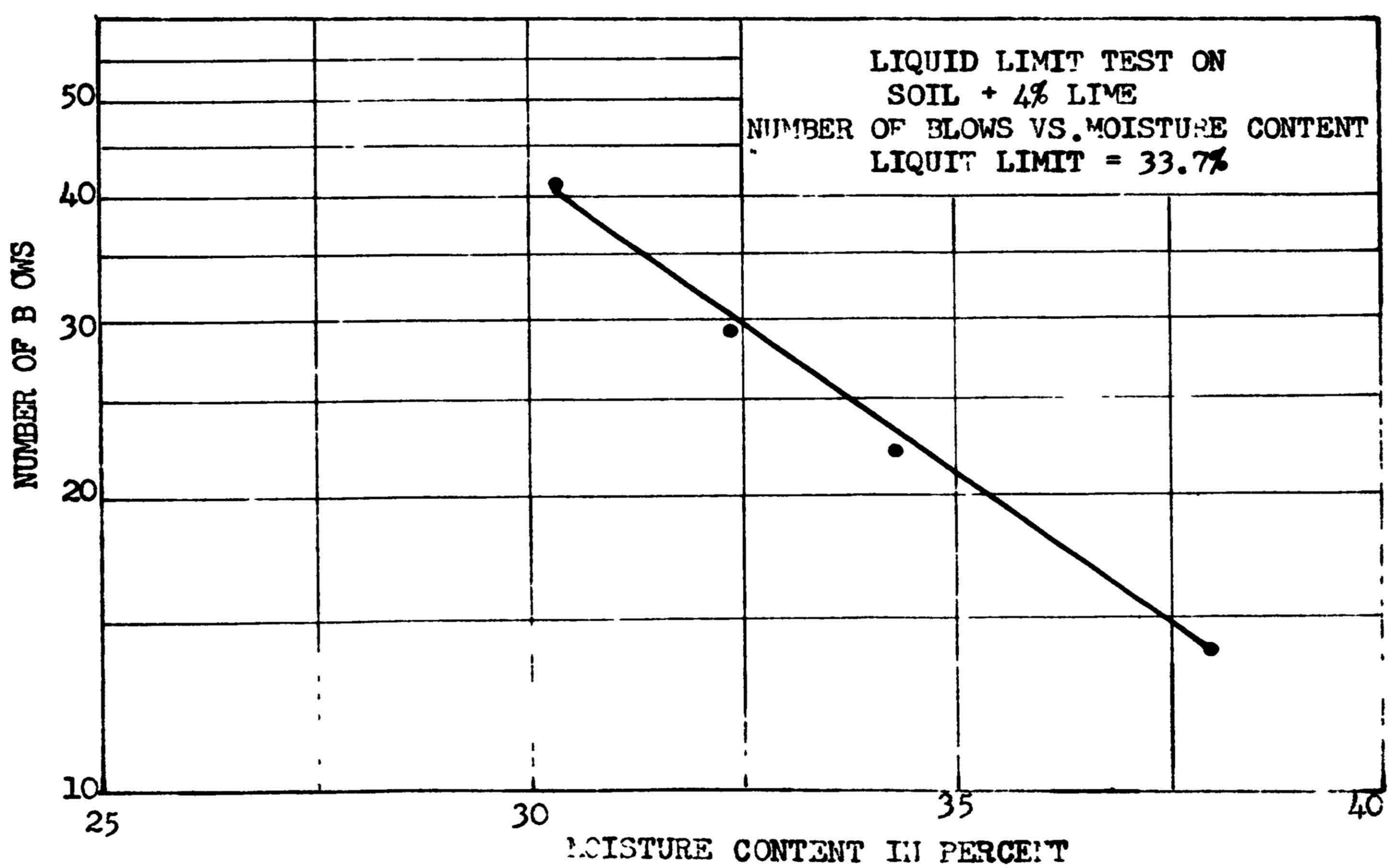
DATA AND GRAPHS OF LIQUID LIMIT TESTS

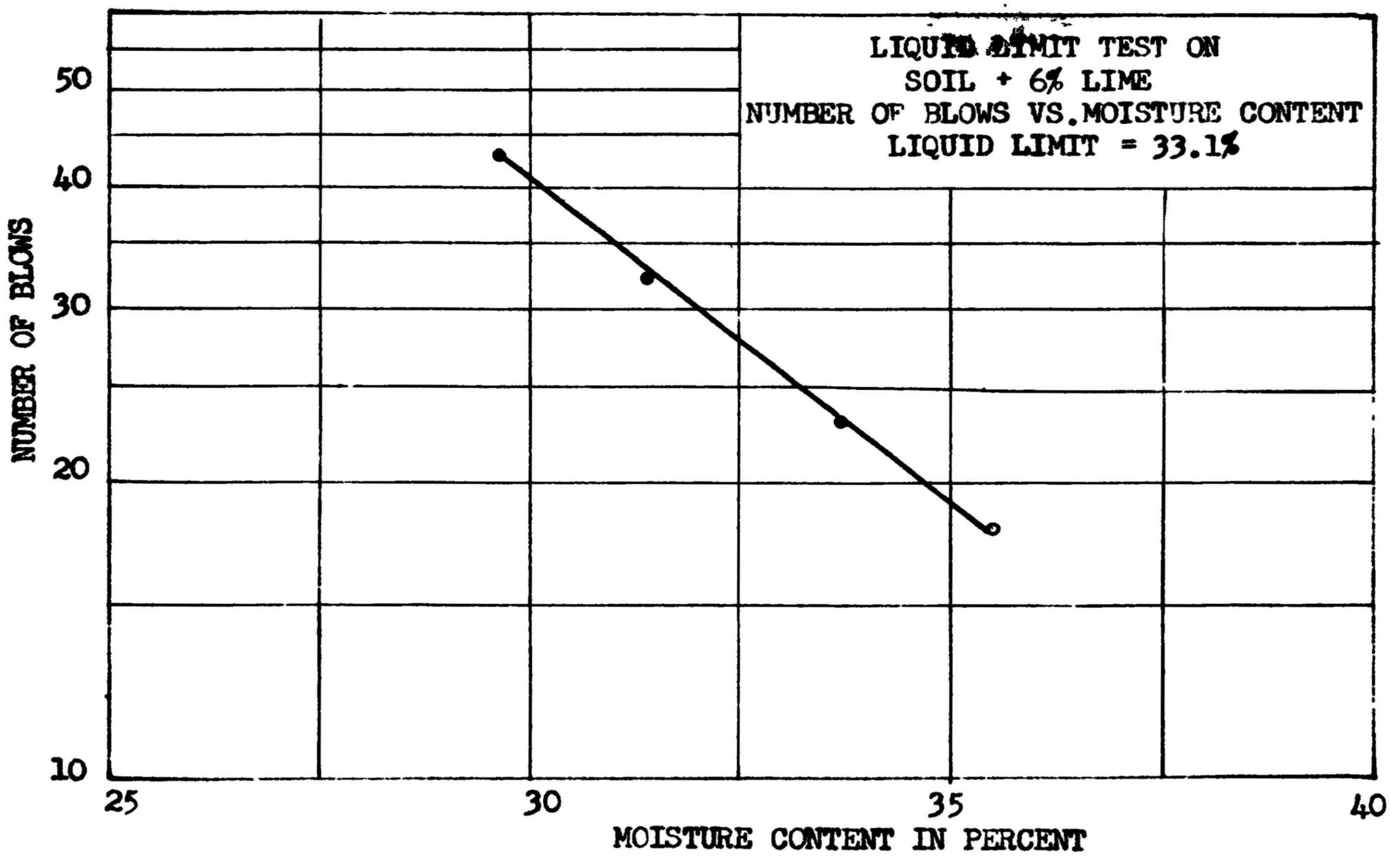
MIXTURE	NUMBER OF BLOWS	MOISTURE CONTENT PERCENT
Natural Soil	35	33.4
	27	35.3
	21	36.4
	14	38.5
Soil + 2% Lime	38	32.0
	29	33.2
	23	34.6
	16	36.8
Soil + 4% Lime	41	30.3
	29	32.4
	22	34.3
	14	38.0
Soil + 6% Lime	43	29.8
	32	31.4
	23	33.7
	18	35.5
Soil + 8% Lime	38	27.4
	27	31.2
	21	33.2
	15	35.5
Soil + 10% Lime	42	26.2
	31	28.6
	22	31.0
	16	33.5

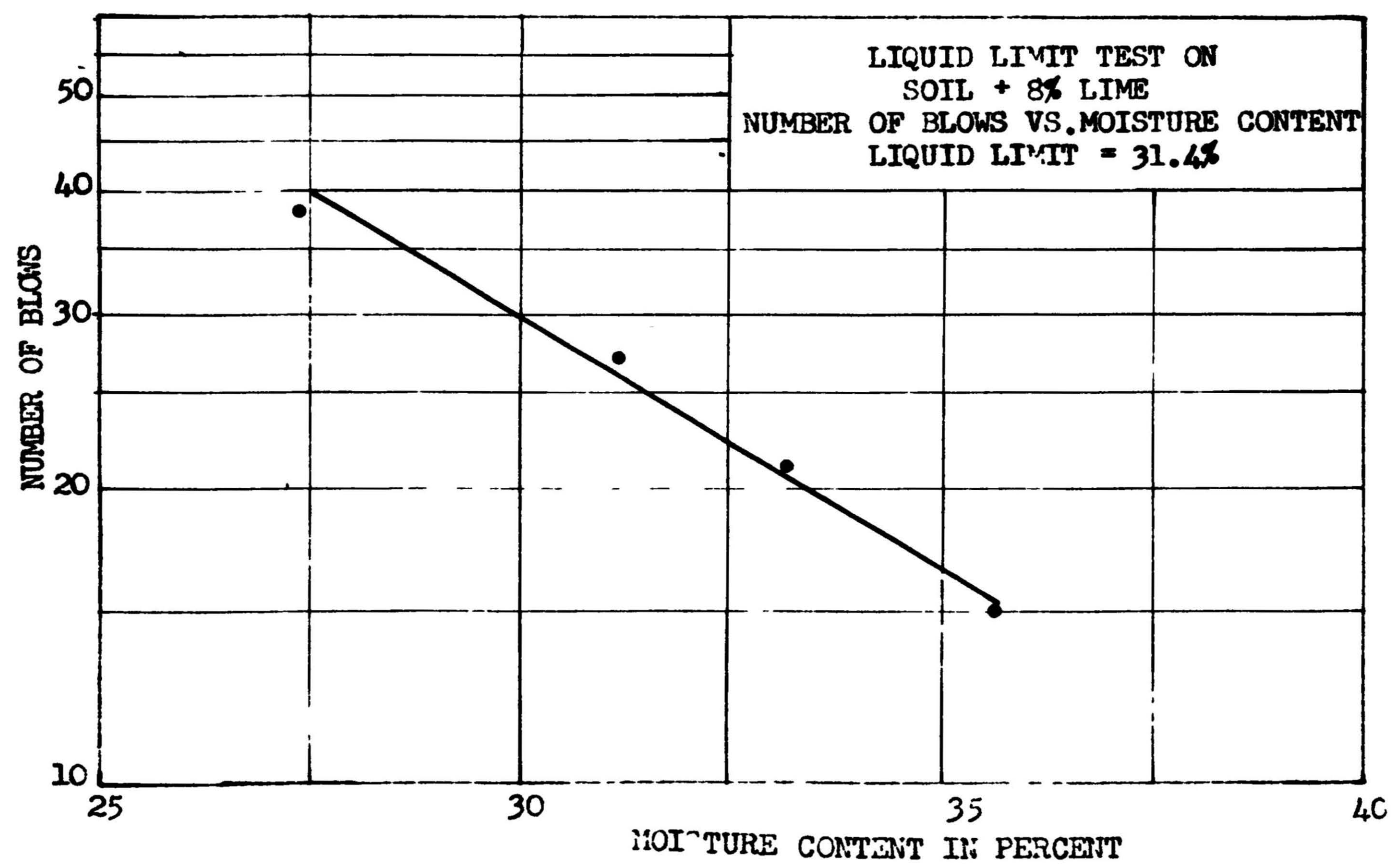
MIXTURE	NUMBER OF BLOWS	MOISTURE CONTENT PERCENT
Soil + 2% Cement	39	30.6
	28	32.5
	21	34.6
	14	37.8
Soil + 4% Cement	41	29.1
	29	32.3
	24	33.3
	17	36.5
Soil + 6% Cement	35	31.0
	26	33.8
	18	36.2
	14	39.0
Soil + 8% Cement	35	29.8
	27	31.8
	20	33.6
	16	35.5
Soil + 10% Cement	37	29.5
	28	31.3
	23	33.5
	17	35.8

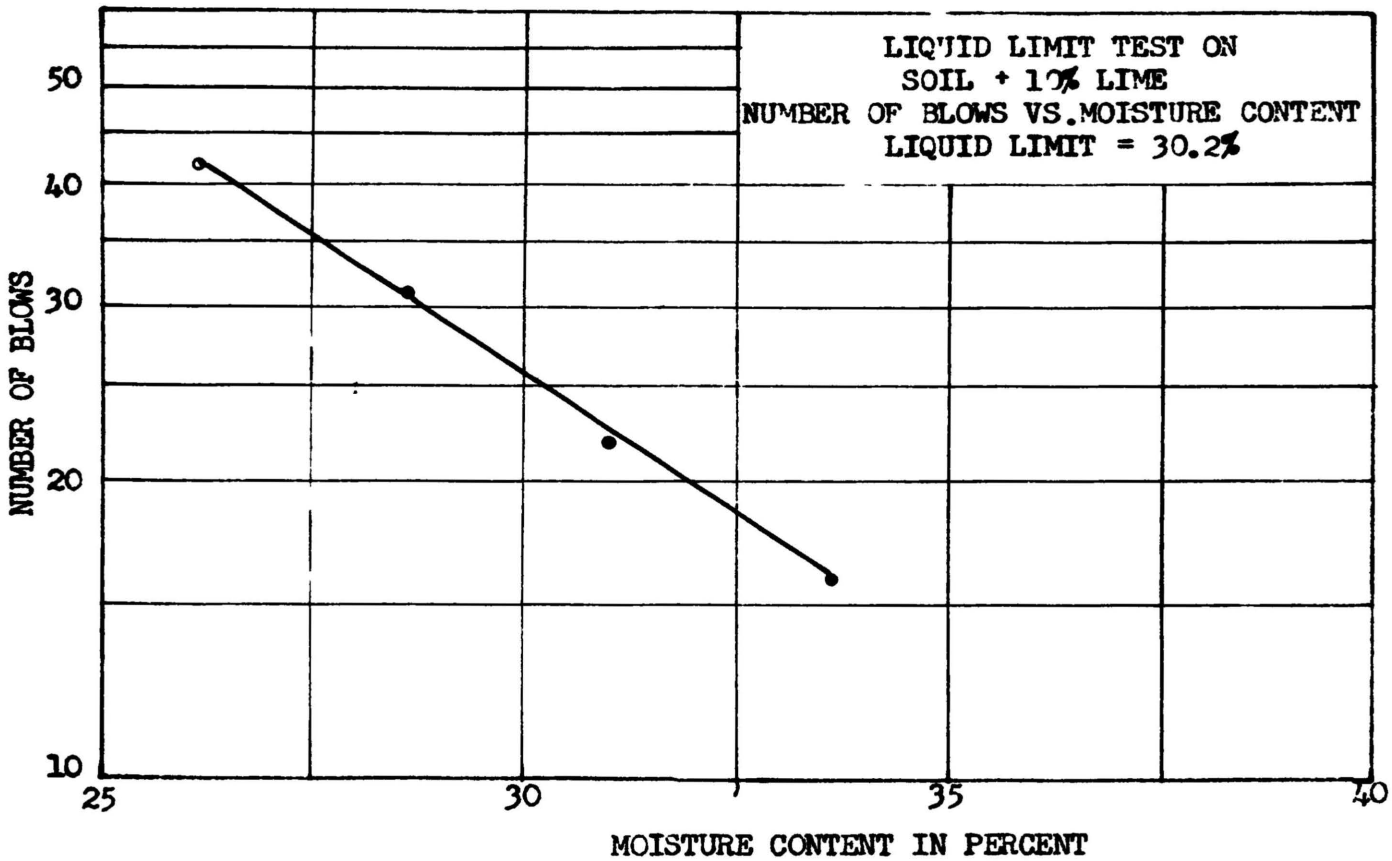

LIQUID LIMIT TEST ON
NATURAL SOIL
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUID LIMIT = 35.5%
NUMBER OF BLOWS
50
40
30
20
10
30
35
40
45
MOISTURE CONTENT IN PERCENT

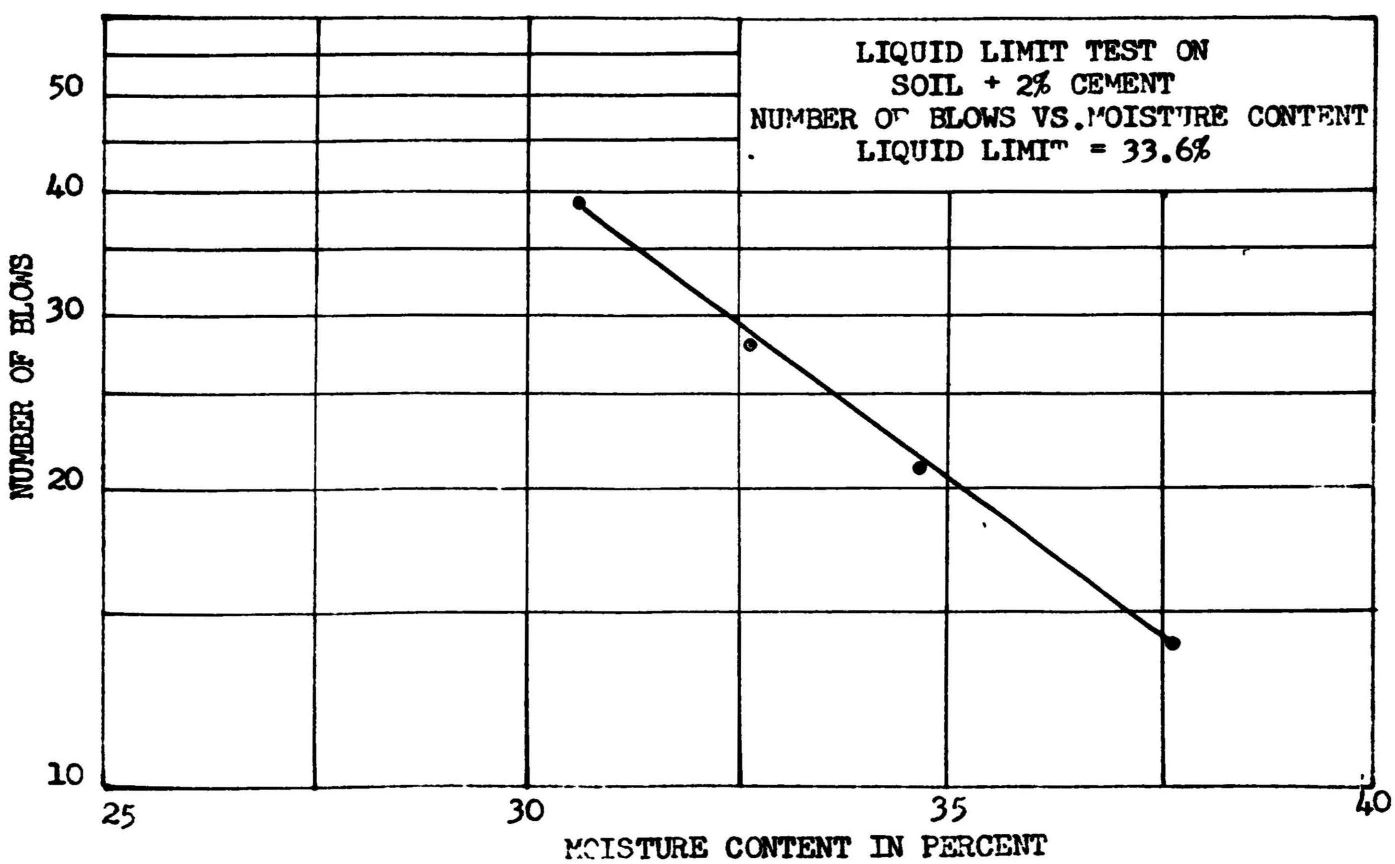

LIQUID LIMIT TEST ON
SOIL + 2% LIME
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUID LIMIT = 34.2%
NUMBER OF BLOWS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT

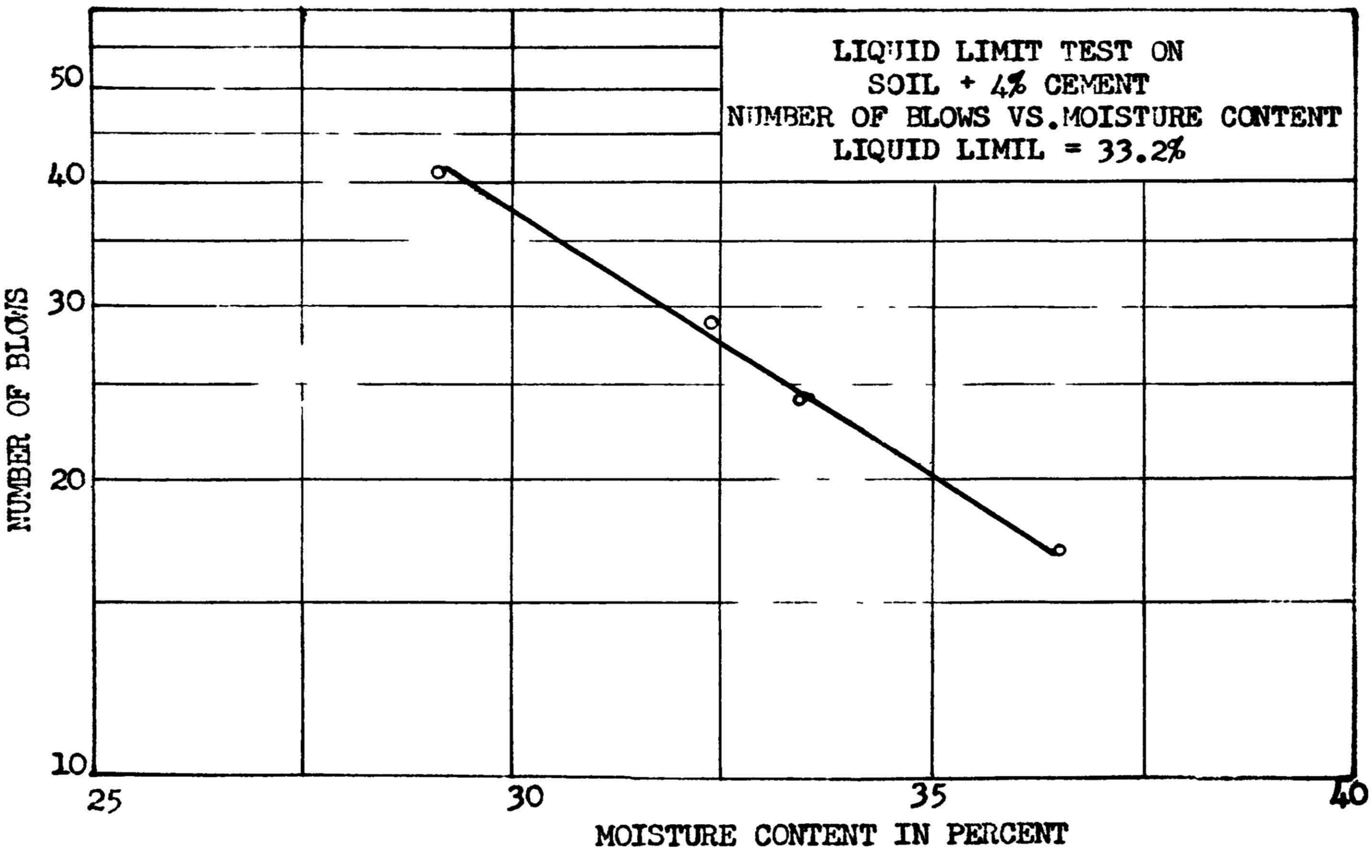

LIQUID LIMIT TEST ON
SOIL + 4% LIME
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUIT LIMIT = 33.7%
NUMBER OF B OWS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT

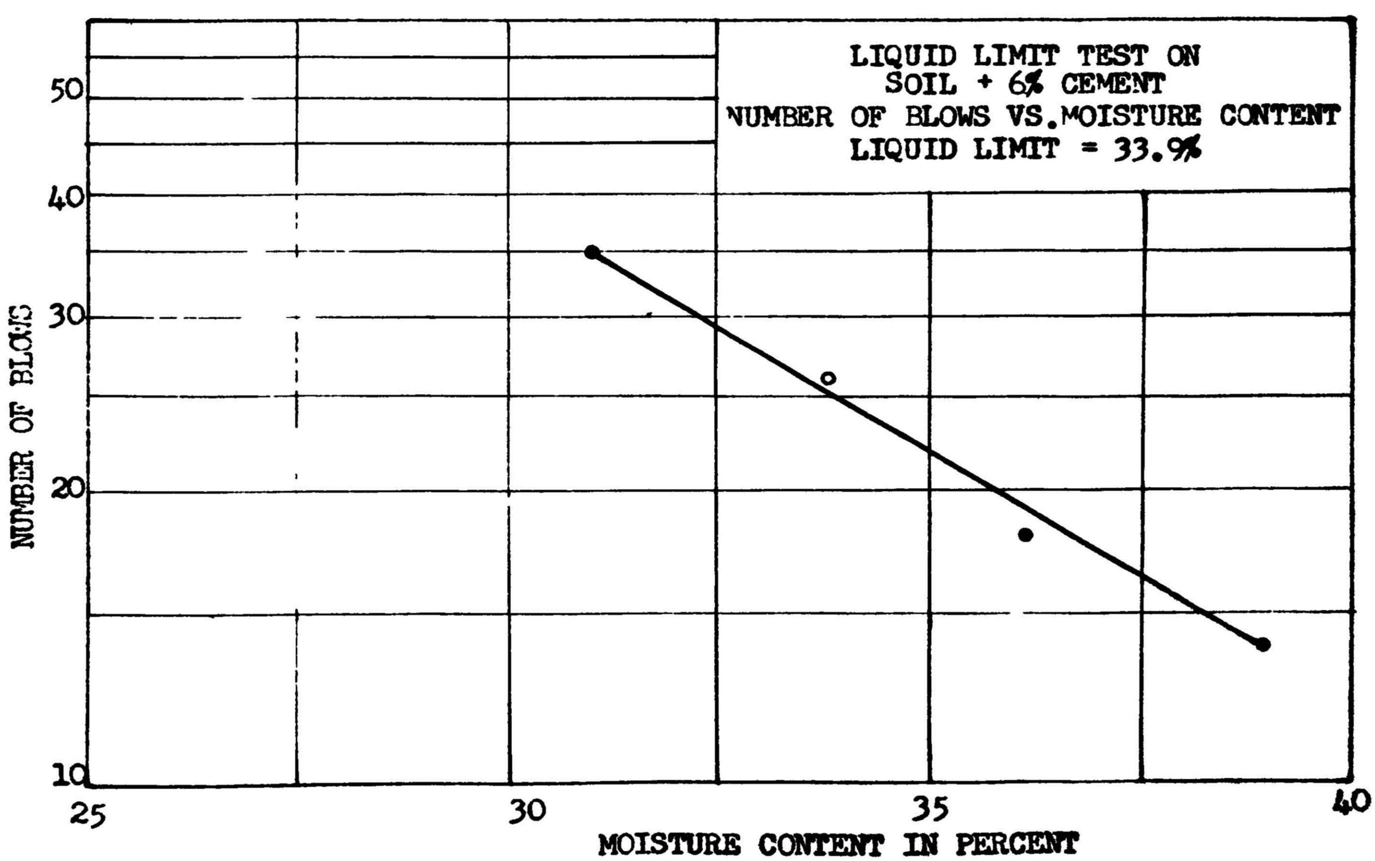

LIQUID LIMIT TEST ON
SOIL + 6% LIME
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUID LIMIT = 33.1%
NUMBER OF BLOWS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT

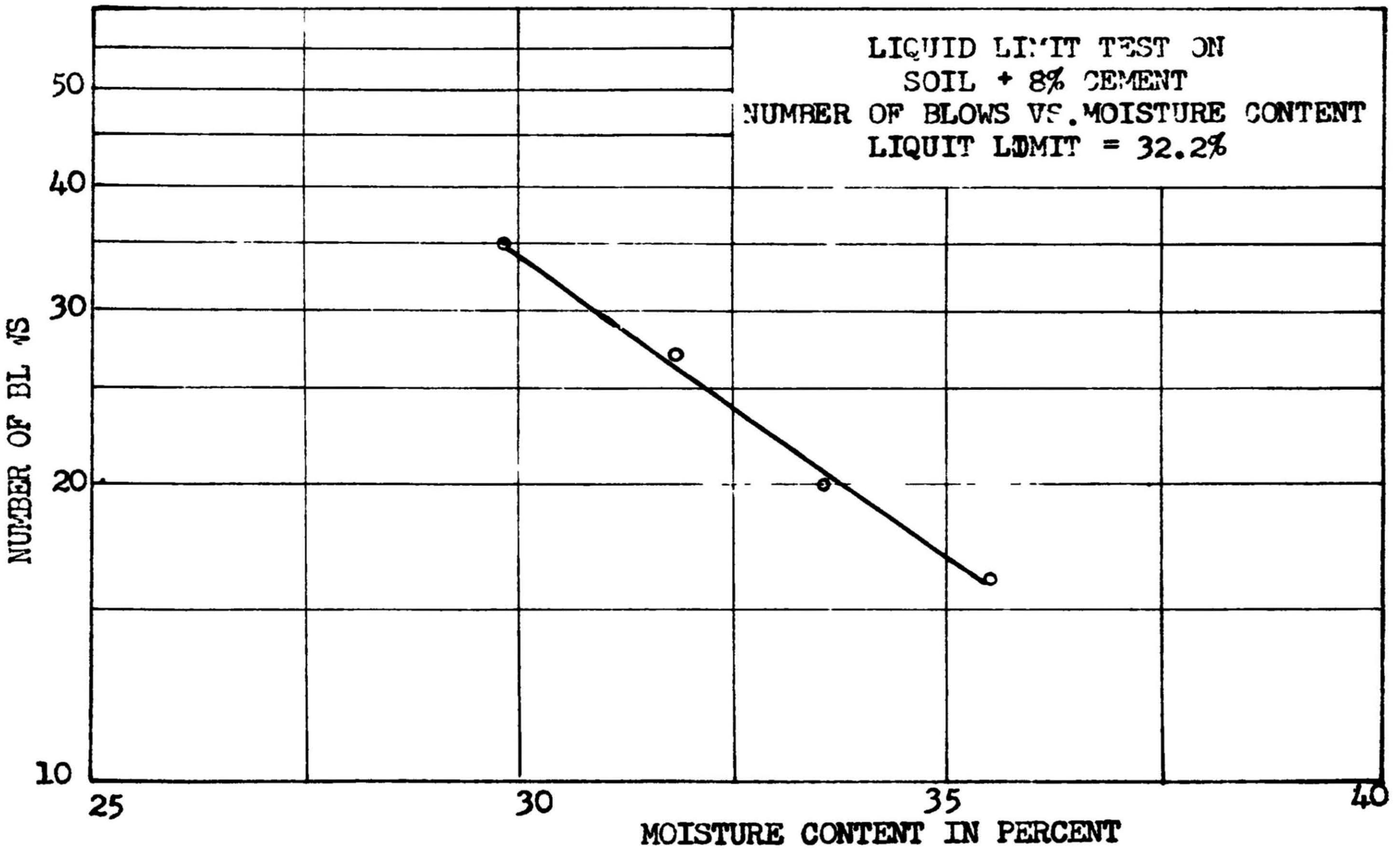

LIQUID LIMIT TEST ON
SOIL + 8% LIME
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUID LIMIT = 31.4%
NUMBER OF BLOWS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT

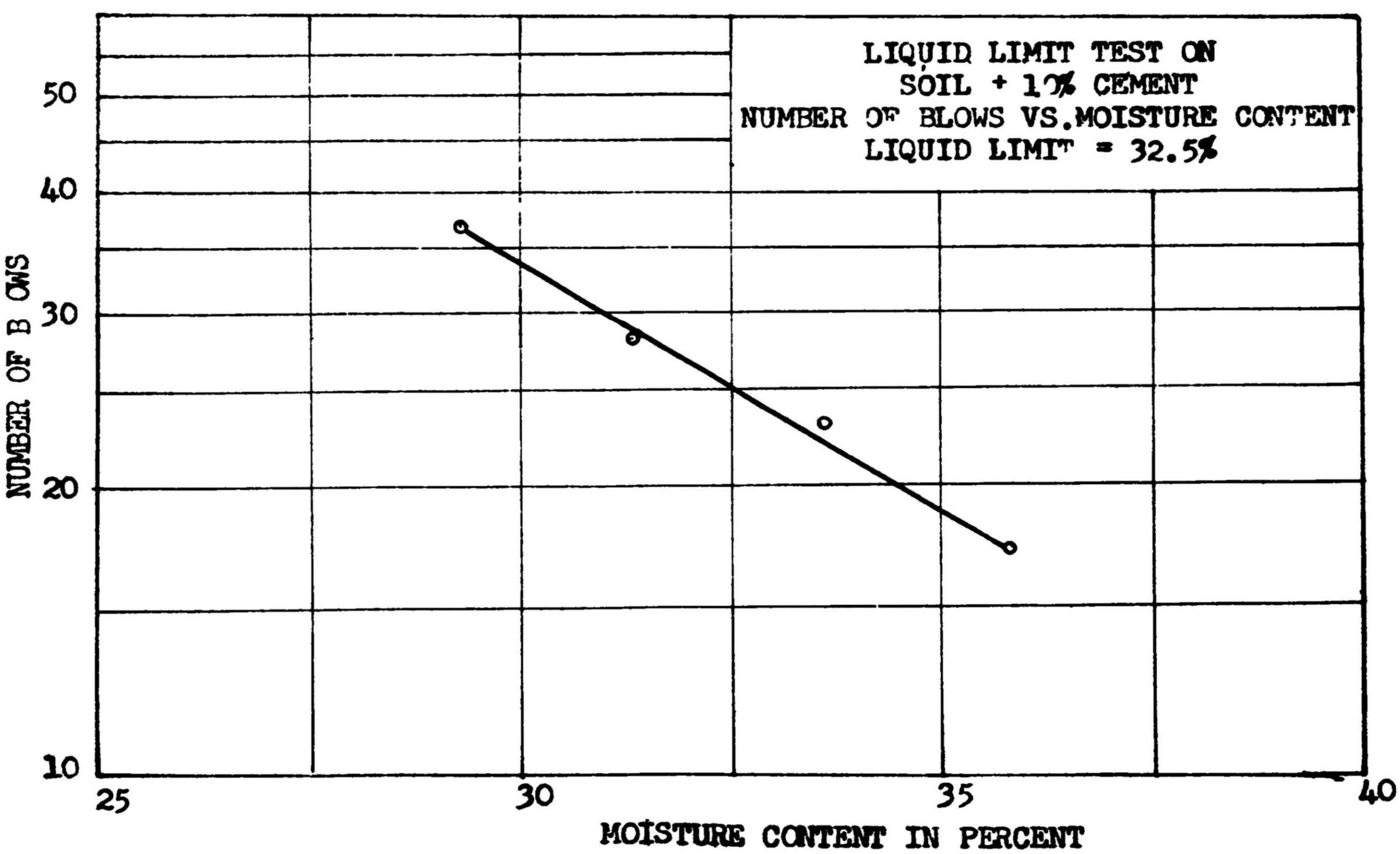

LIQUID LIMIT TEST ON
SOIL + 1% LIME
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUID LIMIT = 30.2%
NUMBER OF BLOWS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT


LIQUID LIMIT TEST ON
SOIL + 2% CEMENT
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUID LIMIT = 33.6%
NUMBER OF BLOWS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT


LIQUID LIMIT TEST ON
SOIL + 4% CEMENT
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUID LIMIL = 33.2%
NUMBER OF BLOWS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT


LIQUID LIMIT TEST ON
SOIL + 6% CEMENT
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUID LIMIT = 33.9%
NUMBER OF BLOWS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT


LIQUID LIMIT TEST ON
SOIL + 8% CEMENT
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUIT LIMIT = 32.2%
NUMBER OF BL WS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT


LIQUID LIMIT TEST ON
SOIL + 10% CEMENT
NUMBER OF BLOWS VS. MOISTURE CONTENT
LIQUID LIMIT = 32.5%
NUMBER OF B OWS
50
40
30
20
10
25
30
35
40
MOISTURE CONTENT IN PERCENT

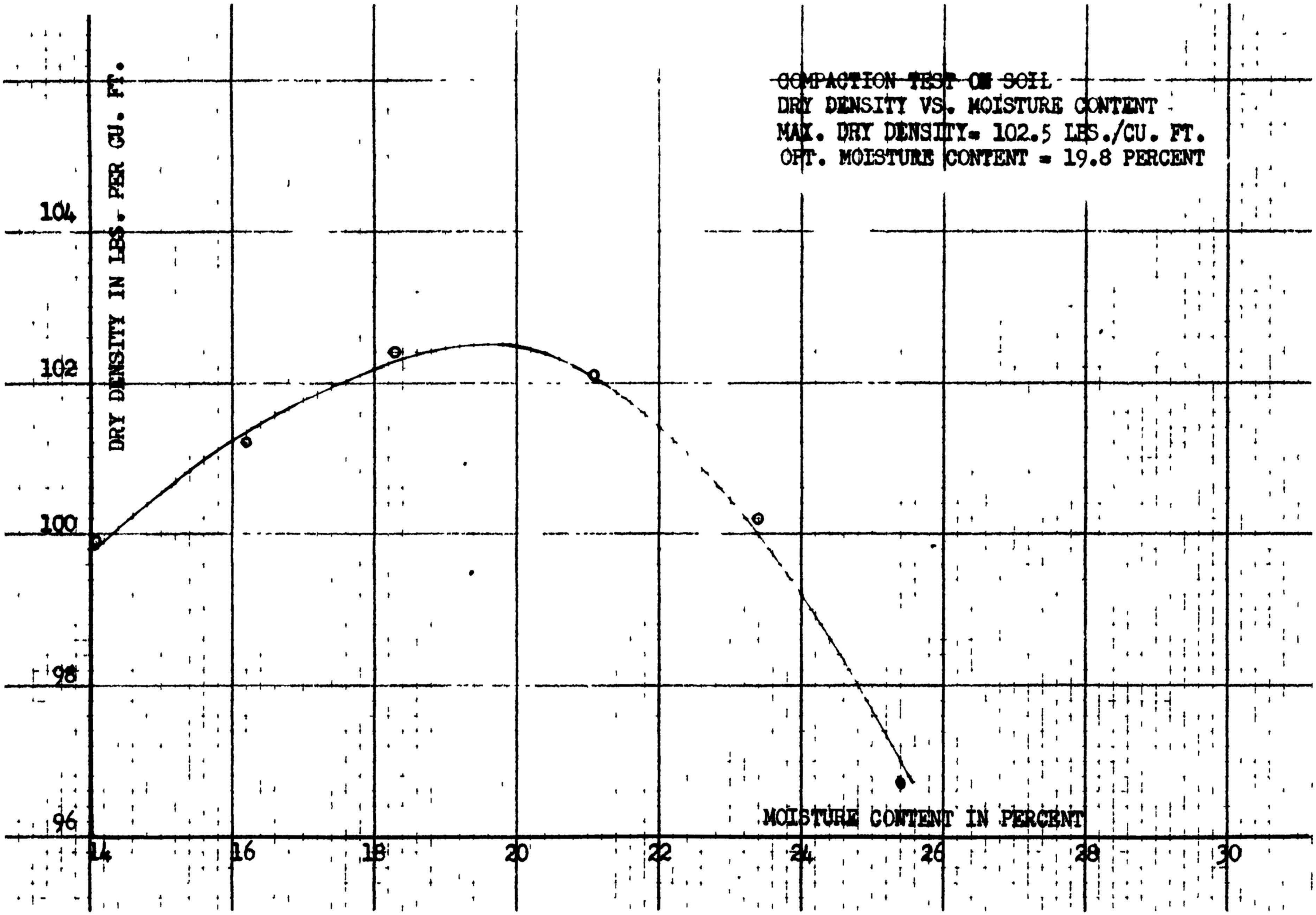
APPENDIX B

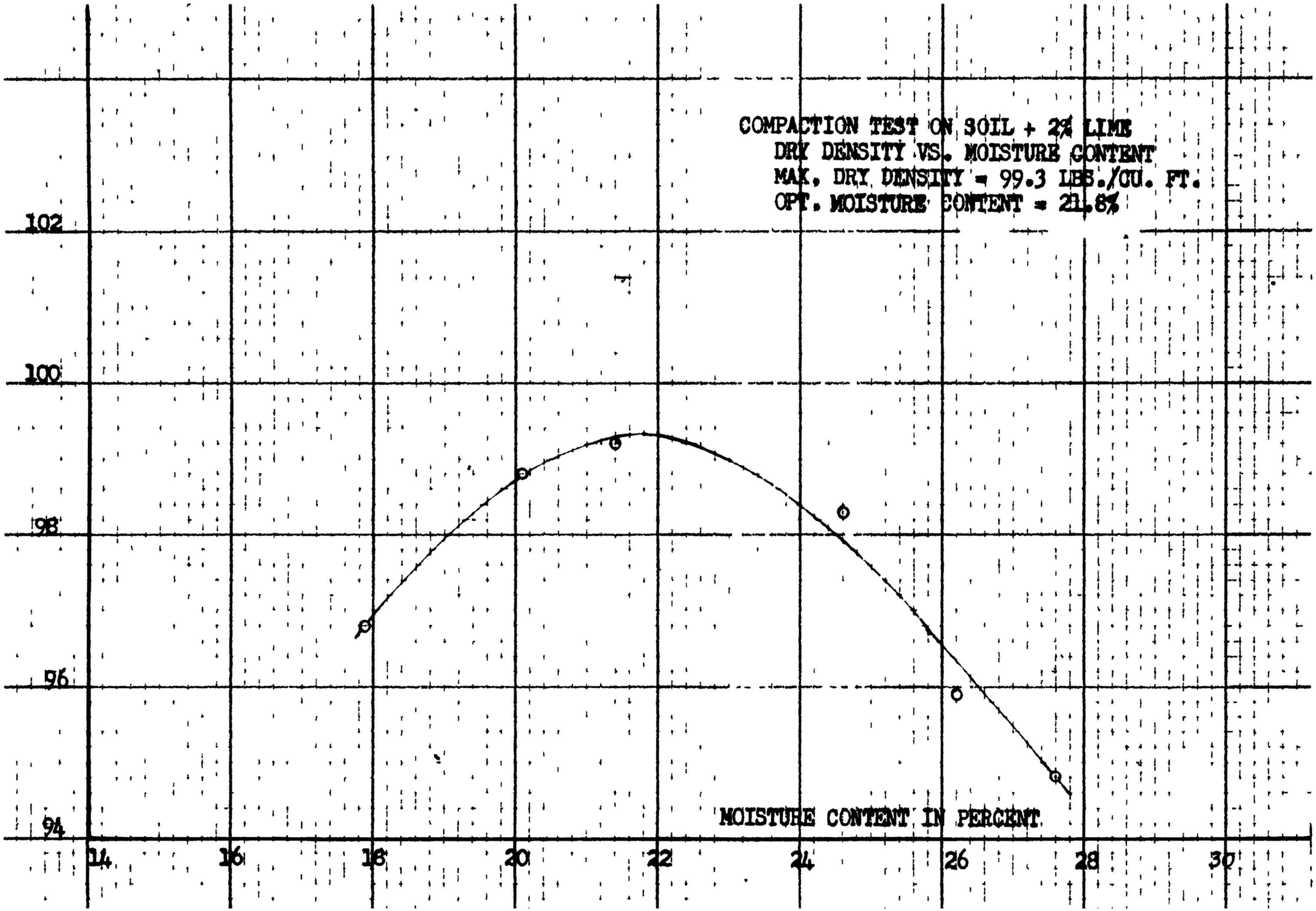
DATA AND GRAPHS OF MOISTURE-DENSITY RELATIONS TESTS

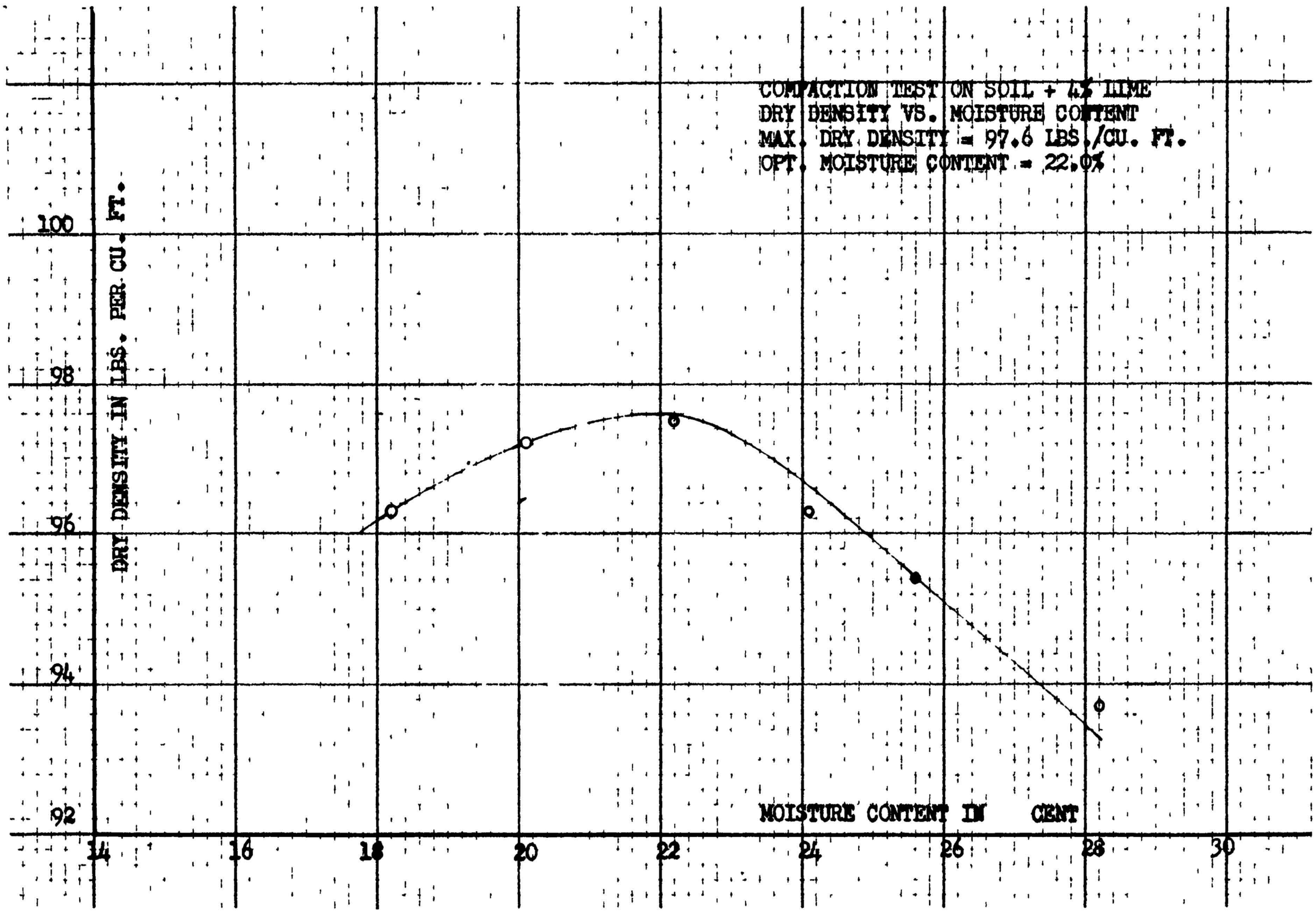
MIXTURE	DRY DENSITY LBS. PER CU. FT.	MOISTURE CONTENT PERCENT
Natural Soil	99.9	14.1
	101.2	16.2
	102.4	18.3
	102.1	21.1
	100.2	23.4
	96.7	25.4
Soil + 2% Lime	96.8	17.9
	98.8	20.1
	99.2	21.4
	98.3	24.6
	95.9	26.2
	94.8	27.6
Soil + 4% Lime	96.3	18.2
	97.2	20.1
	97.5	22.2
	96.3	24.1
	95.4	25.6
	93.7	28.2
Soil + 6% Lime	95.1	18.1
	96.4	19.9
	97.2	22.0
	95.1	26.2
	93.1	28.2
	90.6	30.2

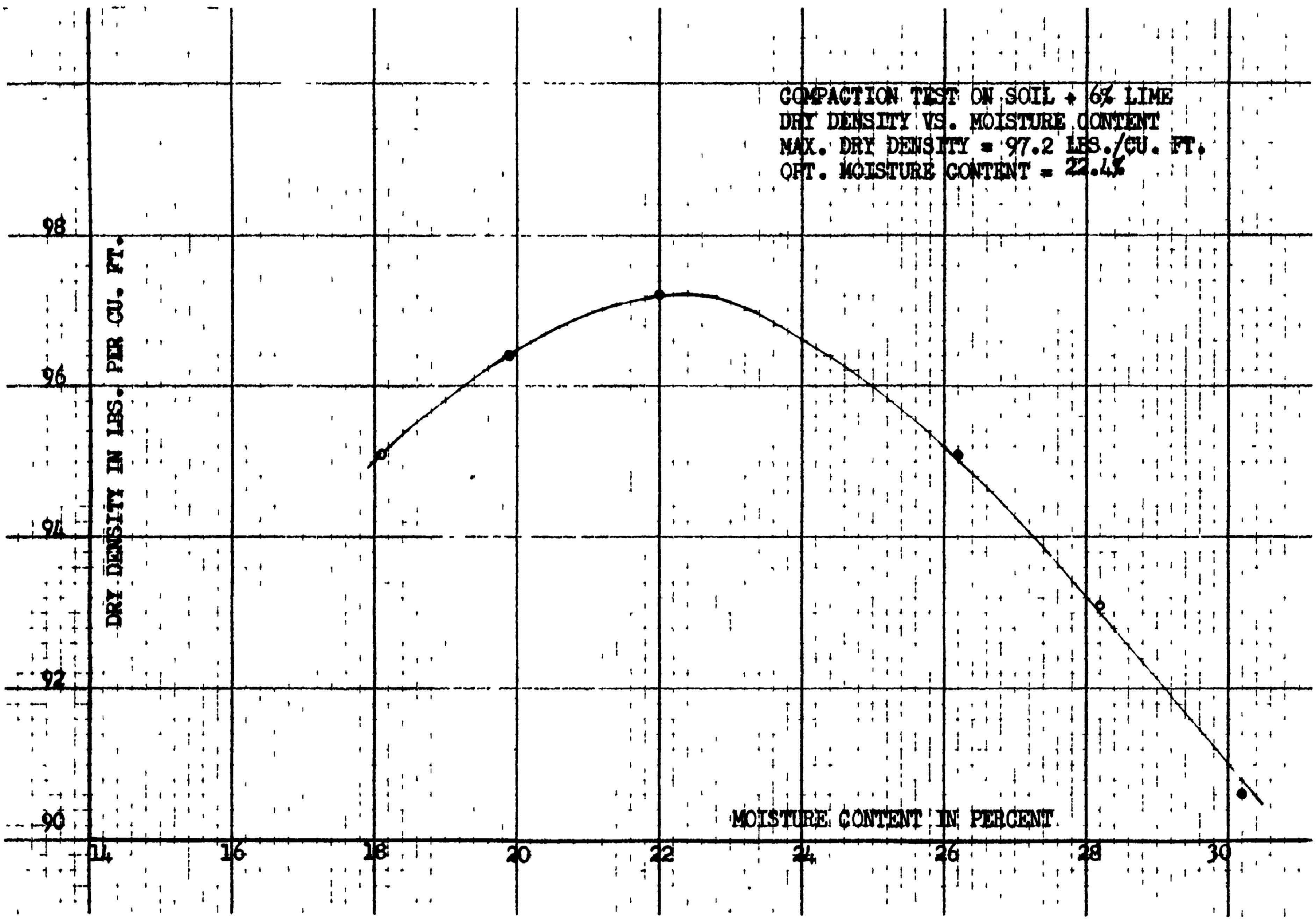
MIXTURE	DRY DENSITY LBS. PER CU. FT.	MOISTURE CONTENT PERCENT
Soil + 8% Lime	93.9	17.5
	95.5	19.8
	96.8	22.3
	96.4	25.3
	94.8	26.9
	92.3	28.9
Soil + 10% Lime	91.4	18.5
	93.1	19.8
	94.2	23.4
	93.7	26.0
	92.3	27.9
	89.9	30.6
Soil + 2% Cement	96.2	16.1
	99.2	18.6
	100.6	20.7
	100.1	23.1
	98.2	25.8
	95.8	28.3
Soil + 4% Cement	97.9	14.4
	99.2	16.9
	100.3	19.6
	99.7	22.1
	97.4	24.2
	94.3	26.4

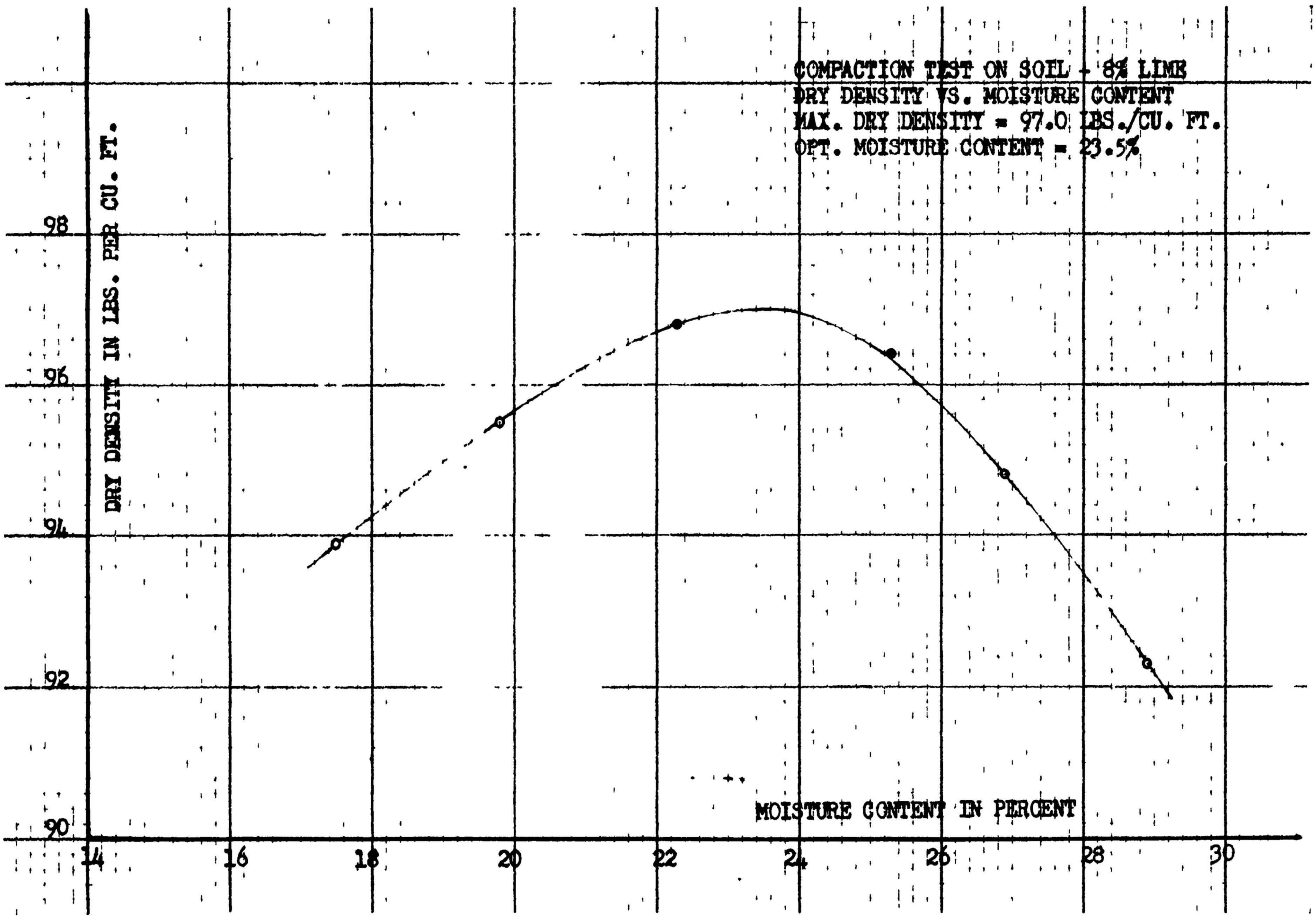
MIXTURE	DRY DENSITY LBS. PER CU. FT.	MOISTURE CONTENT PERCENT
Soil + 6% Cement	95.9	14.8
	99.7	17.2
	102.1	19.6
	102.0	22.4
	100.8	24.8
	98.8	27.4
Soil + 8% Cement	97.2	14.3
	99.9	16.5
	101.5	18.7
	101.4	21.1
	99.8	23.7
	96.8	25.9
Soil + 10% Cement	99.8	15.2
	101.8	17.8
	103.0	20.2
	102.7	22.5
	99.8	24.2
	97.4	25.8

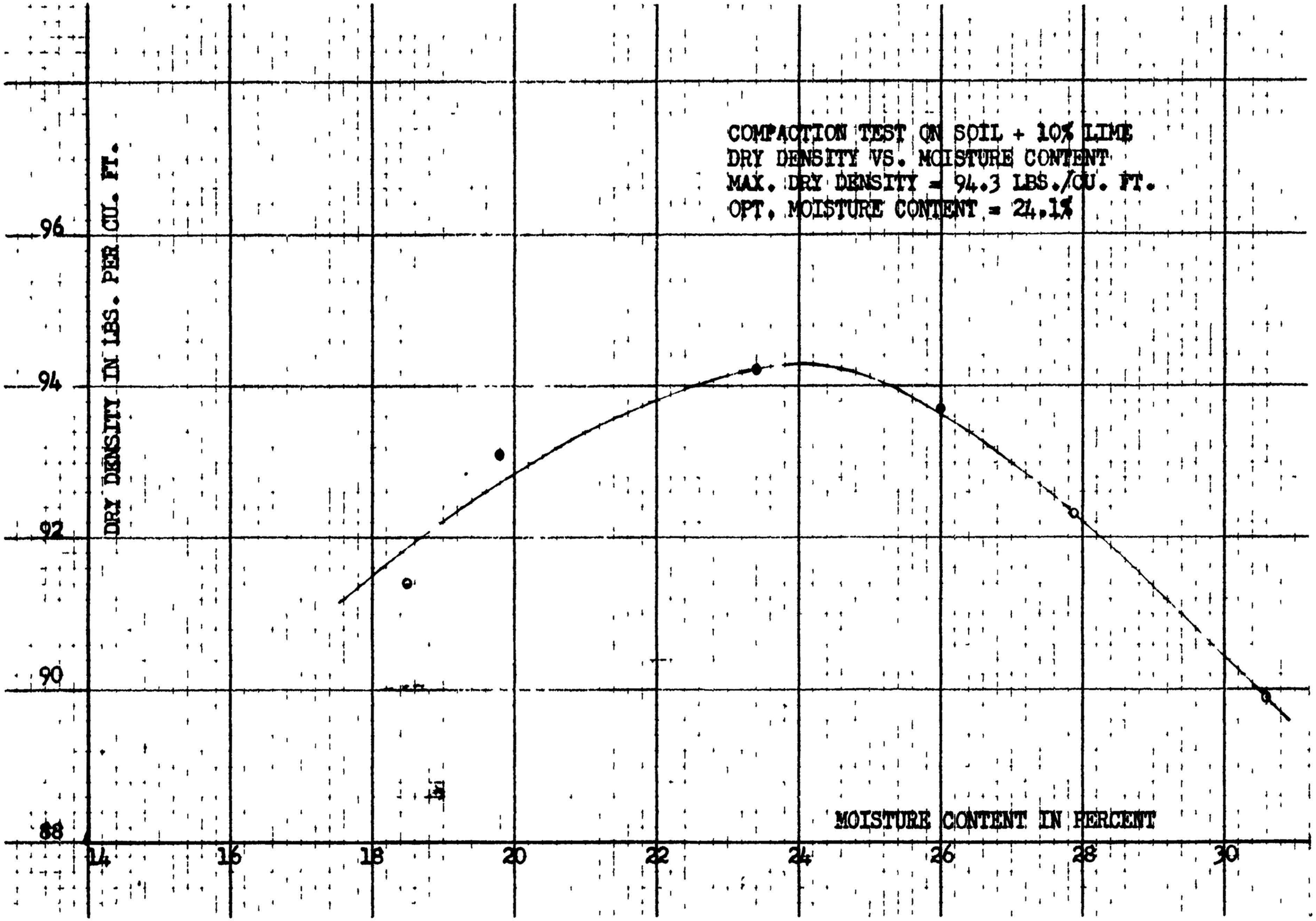

COMPACTION TEST ON SOIL
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 102.5 LBS./CU. FT.
OPT. MOISTURE CONTENT = 19.8 PERCENT
DRY DENSITY IN LBS. PER CU. FT.
MOISTURE CONTENT IN PERCENT
104
102
100
98
96
14
16
18
20
22
24
26
28
30

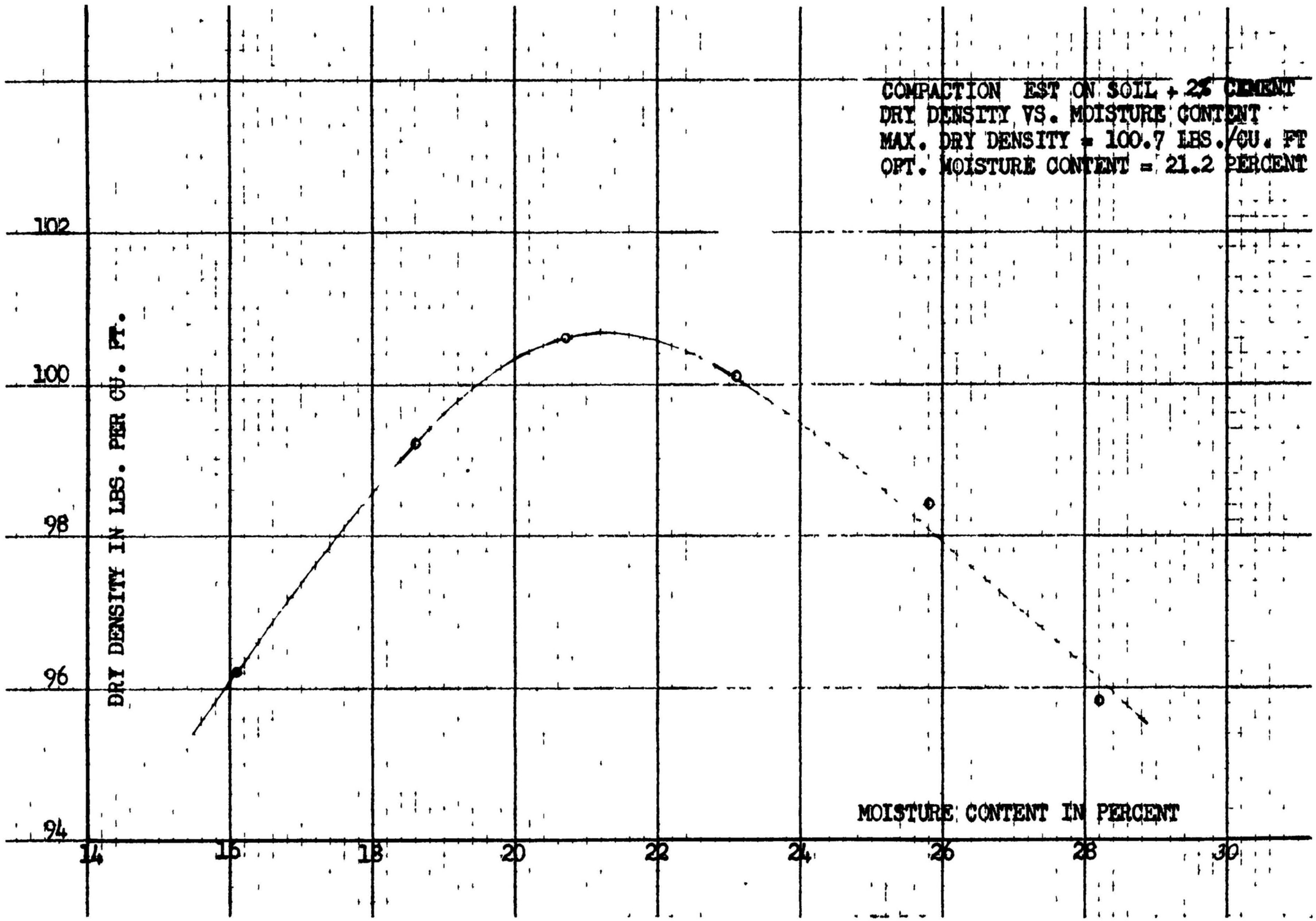

COMPACTION TEST ON SOIL + 2% LIME
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 99.3 LBS./CU. FT.
OPT. MOISTURE CONTENT = 21.8%
102
100
98
96
94
14
16
18
20
22
24
26
28
30
MOISTURE CONTENT IN PERCENT

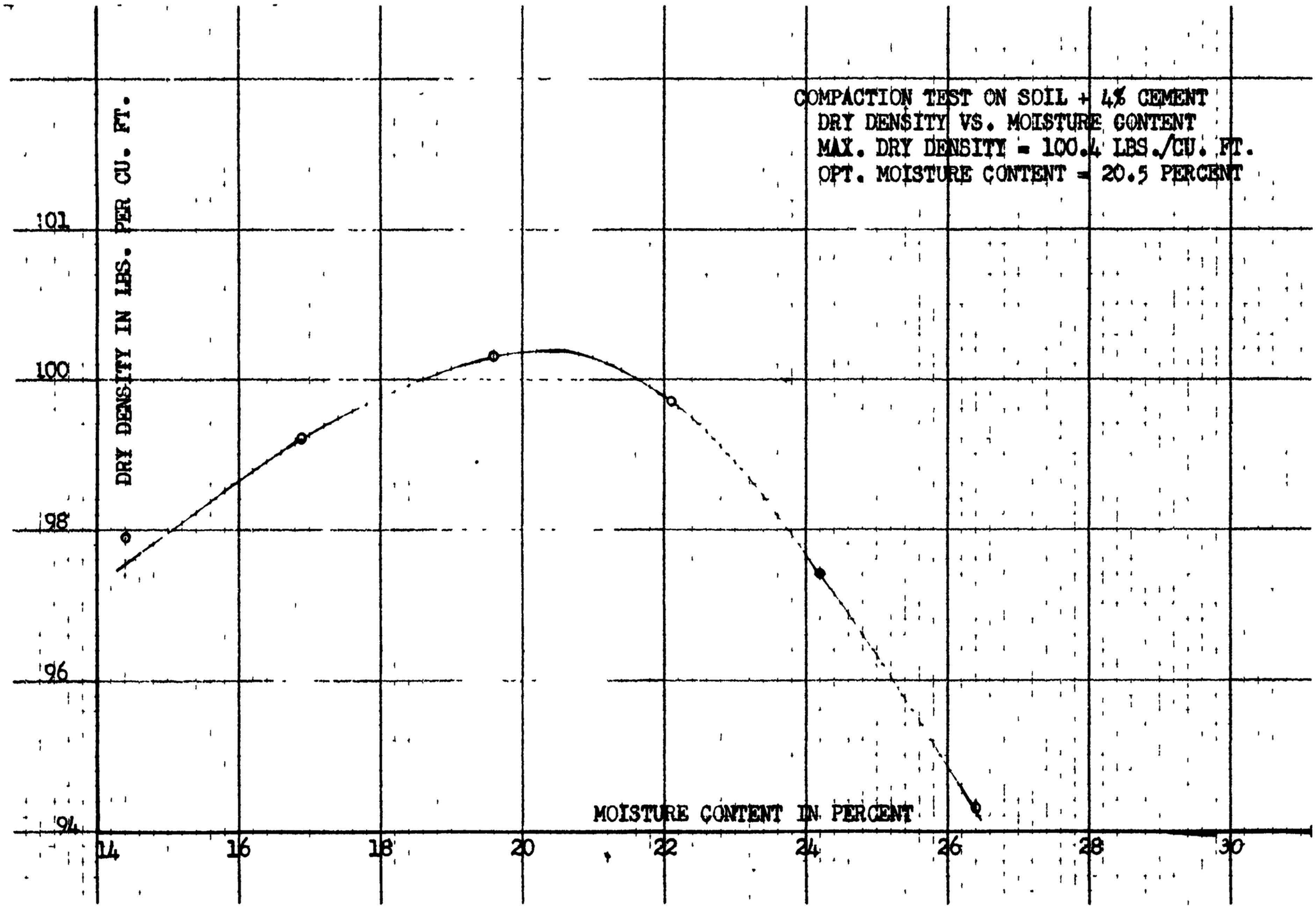

COMPACTION TEST ON SOIL + 4% LIME
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 97.6 LBS./CU. FT.
OPT. MOISTURE CONTENT = 22.0%
DRY DENSITY IN LBS. PER CU. FT.
100
98
96
94
92
MOISTURE CONTENT IN CENT
14
16
18
20
22
24
26
28
30

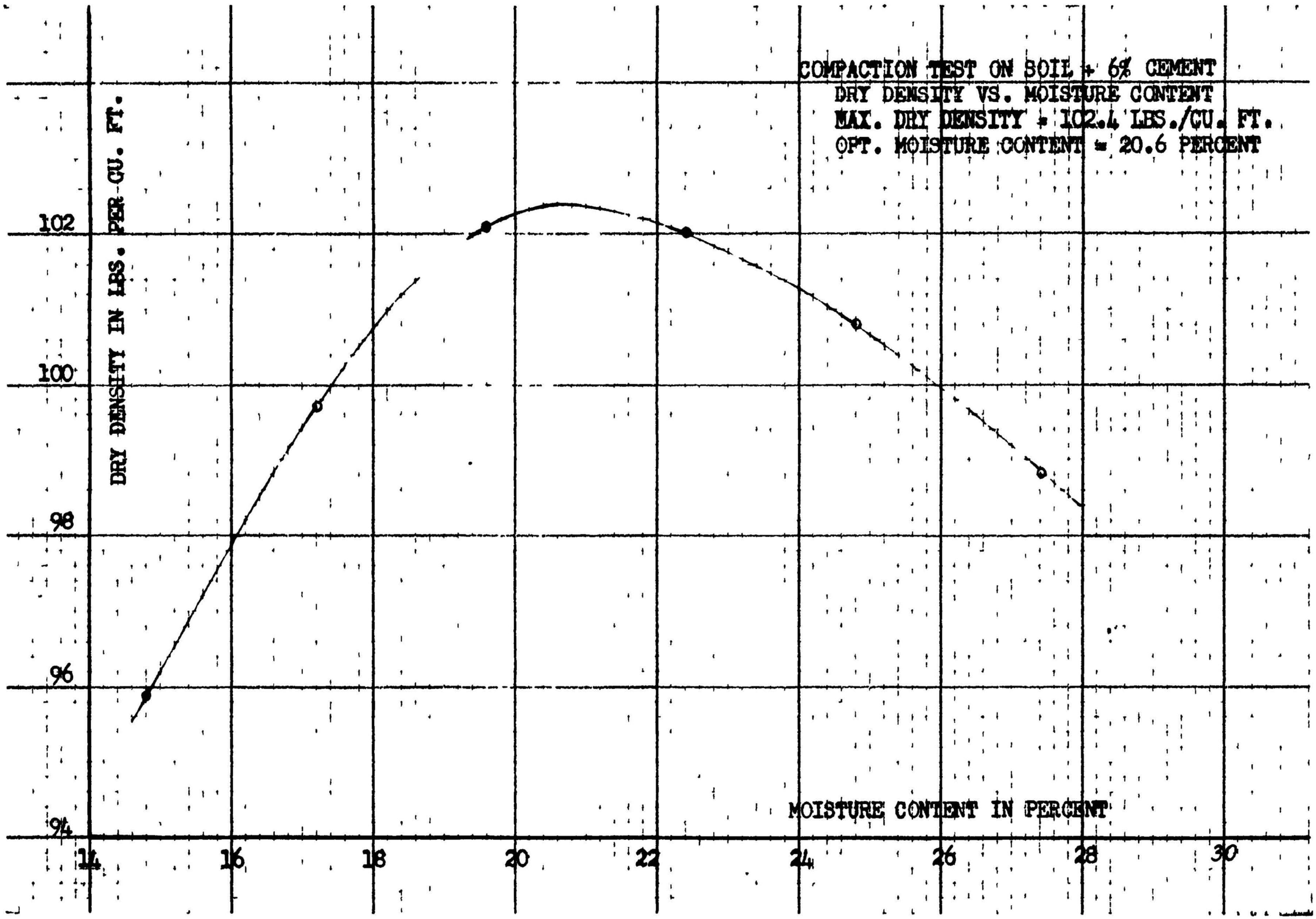

COMPACTION TEST ON SOIL + 6% LIME
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 97.2 LBS./CU. FT.
OPT. MOISTURE CONTENT = 22.4%
DRY DENSITY IN LBS. PER CU. FT.
98
96
94
92
90
MOISTURE CONTENT IN PERCENT
14
16
18
20
22
24
26
28
30

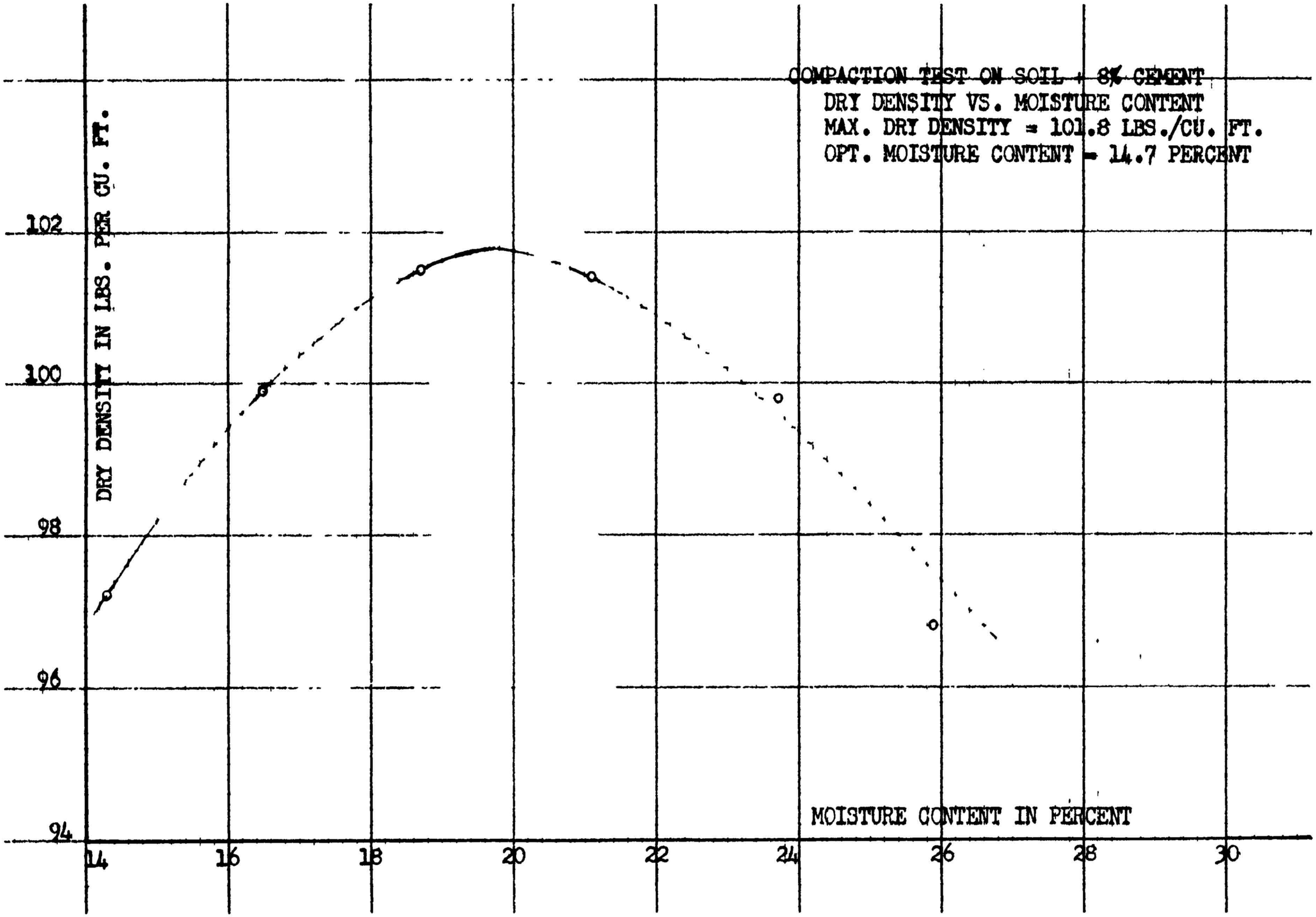

COMPACTION TEST ON SOIL + 8% LIME
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 97.0 LBS./CU. FT.
OPT. MOISTURE CONTENT = 23.5%
DRY DENSITY IN LBS. PER CU. FT.
MOISTURE CONTENT IN PERCENT
90
92
94
96
98
14
16
18
20
22
24
26
28
30

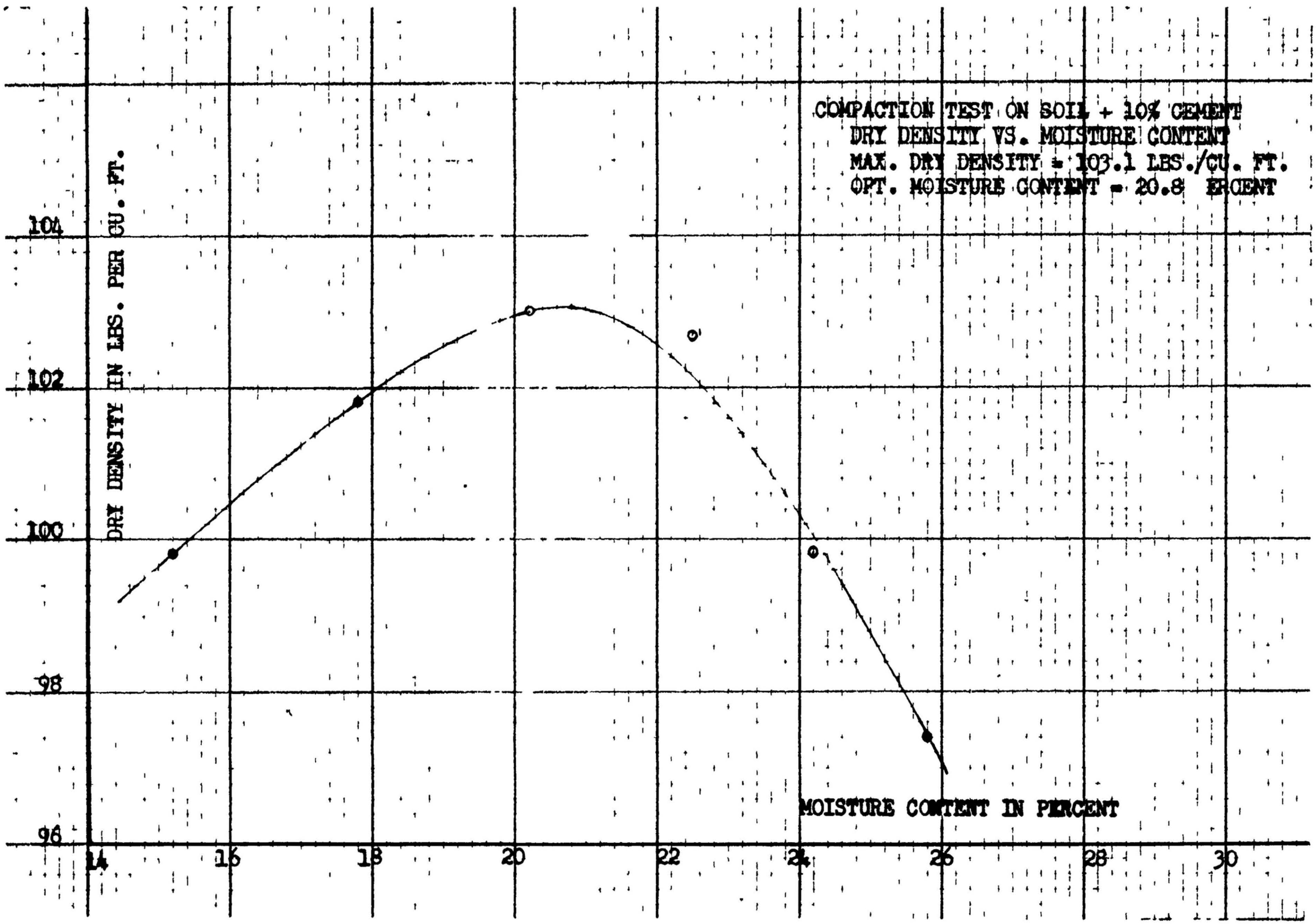

COMPACTION TEST ON SOIL + 10% LIME
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 94.3 LBS./CU. FT.
OPT. MOISTURE CONTENT = 24.1%
DRY DENSITY IN LBS. PER CU. FT.
96
94
92
90
88
MOISTURE CONTENT IN PERCENT
14
16
18
20
22
24
26
28
30


COMPACTION TEST ON SOIL + 2% CEMENT
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 100.7 LBS./CU. FT
OPT. MOISTURE CONTENT = 21.2 PERCENT
DRY DENSITY IN LBS. PER CU. FT.
MOISTURE CONTENT IN PERCENT
102
100
98
96
94
14
16
18
20
22
24
26
28
30


COMPACTION TEST ON SOIL + 4% CEMENT
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 100.4 LBS./CU. FT.
OPT. MOISTURE CONTENT = 20.5 PERCENT
DRY DENSITY IN LBS. PER CU. FT.
MOISTURE CONTENT IN PERCENT
101
100
98
96
94
14
16
18
20
22
24
26
28
30


COMPACTION TEST ON SOIL + 6% CEMENT
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 102.4 LBS./CU. FT.
OPT. MOISTURE CONTENT = 20.6 PERCENT
DRY DENSITY IN LBS. PER CU. FT.
MOISTURE CONTENT IN PERCENT
102
100
98
96
94
14
16
18
20
22
24
26
28
30


COMPACTION TEST ON SOIL + 8% CEMENT
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 101.8 LBS./CU. FT.
OPT. MOISTURE CONTENT = 14.7 PERCENT
DRY DENSITY IN LBS. PER CU. FT.
MOISTURE CONTENT IN PERCENT
102
100
98
96
94
14
16
18
20
22
24
26
28
30


COMPACTION TEST ON SOIL + 10% CEMENT
DRY DENSITY VS. MOISTURE CONTENT
MAX. DRY DENSITY = 103.1 LBS./CU. FT.
OPT. MOISTURE CONTENT = 20.8 PERCENT
DRY DENSITY IN LBS. PER CU. FT.
MOISTURE CONTENT IN PERCENT
104
102
100
98
96
14
16
18
20
22
24
26
28
30

APPENDIX C

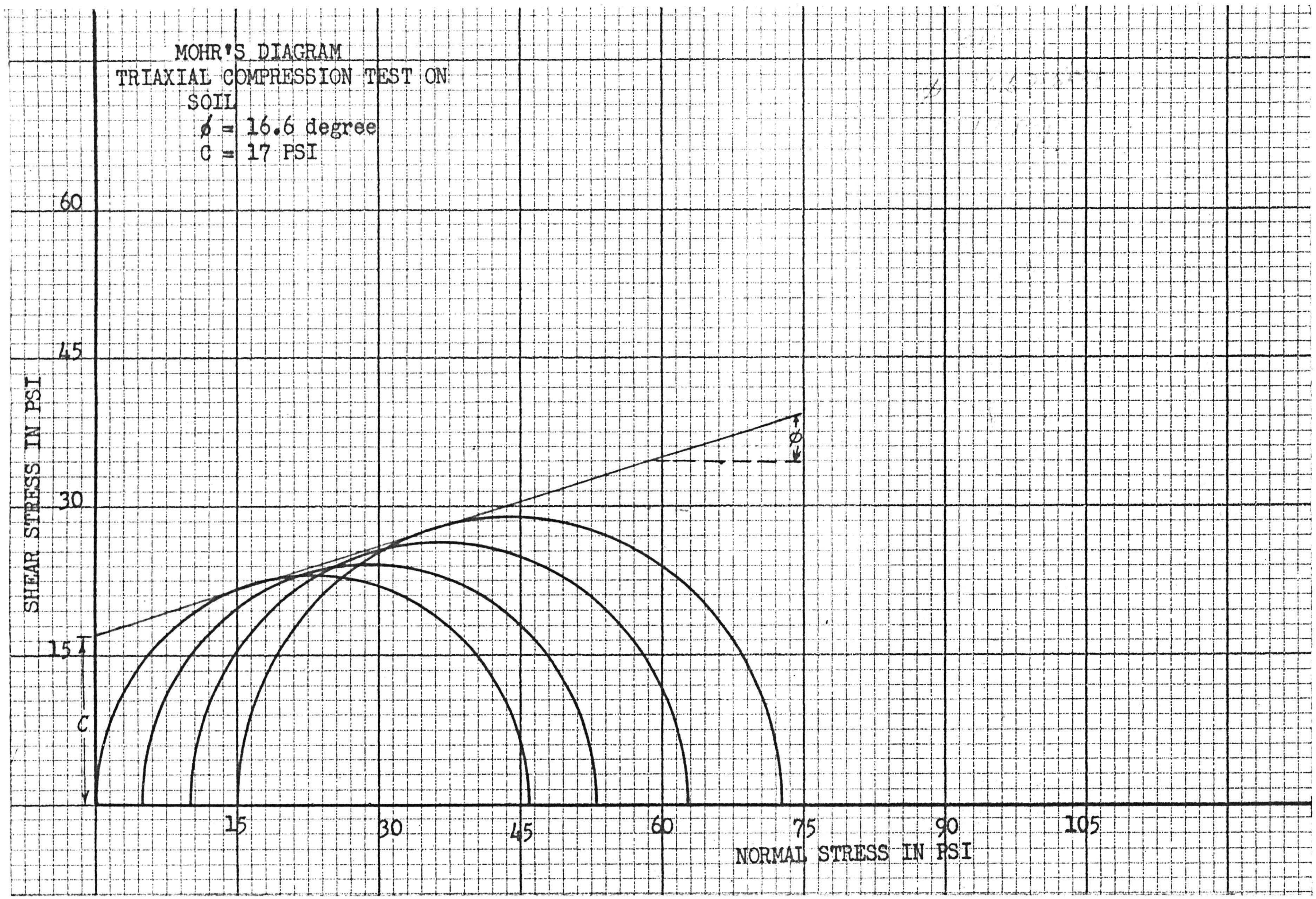
DATA OF CONFINED COMPRESSION TESTS AND MOHR'S CIRCLES OF STRESSES

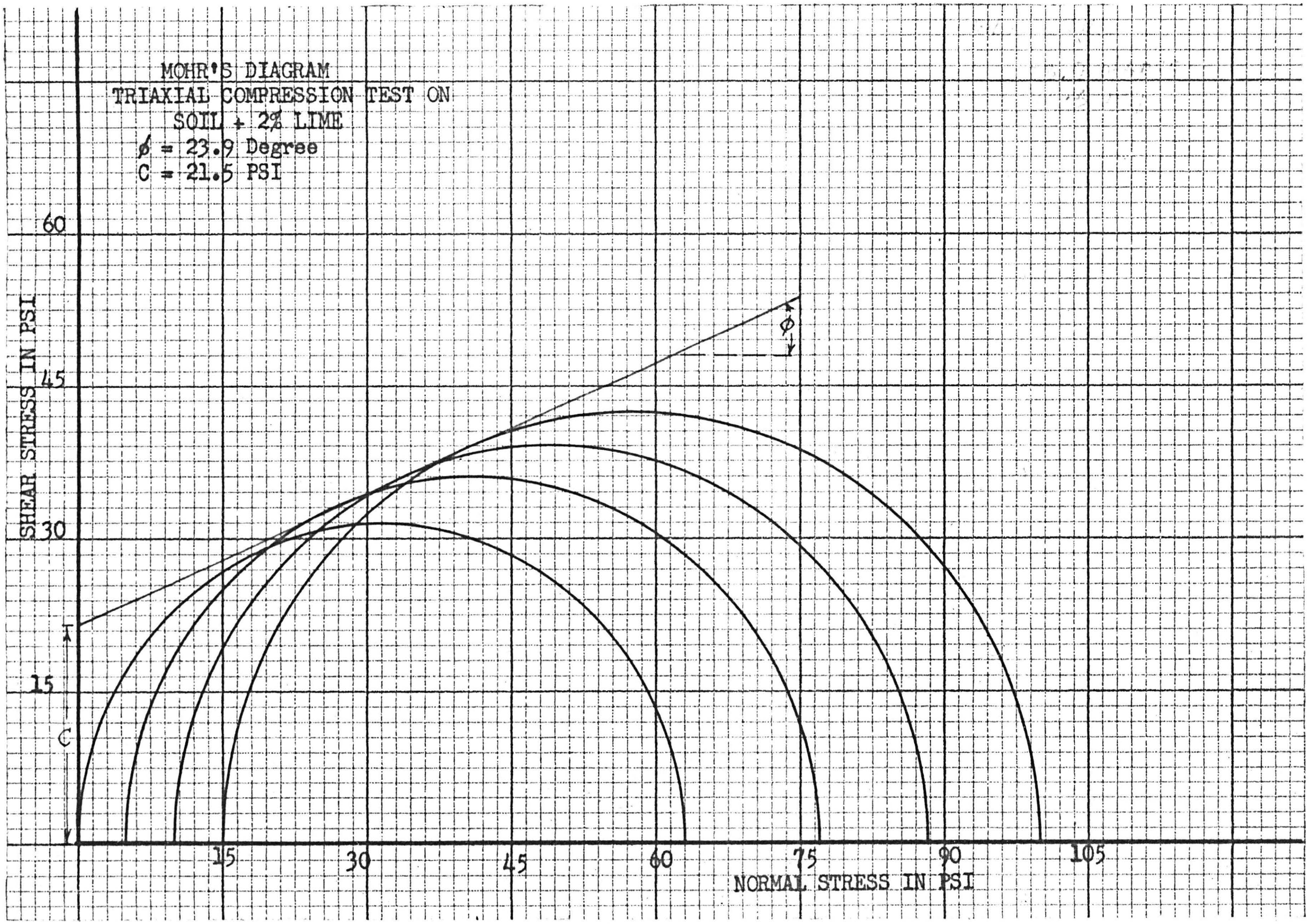
MIXTURE	LATERAL PRESSURE σ_1 P.S.I.	AVERAGE DIAL READING AT FAILURE 10^{-4} IN.	LOAD AT FAILURE LBS.	*ULTIMATE STRESS σ_3 P.S.I.	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\sigma_1 + \sigma_3}{2}$
Natural Soil	0	309	284	46.1	23.1	3.1
	5	321	295	52.9	24.0	29.0
	10	352	324	62.6	26.3	36.3
	15	386	356	72.8	28.9	43.9
Soil + 2% Lime	0	415	388	63.0	31.5	31.5
	5	452	442	76.8	35.9	40.9
	10	471	482	88.3	39.2	49.2
	15	490	523	99.9	42.4	57.4
Soil + 4% Lime	0	463	465	75.5	37.8	37.8
	5	530	606	103.4	49.2	54.2
	10	564	676	120.1	55.1	65.1
	15	590	735	134.4	59.7	74.7
Soil + 6% Lime	0	512	568	92.2	46.1	46.1
	5	566	683	115.9	55.4	60.4
	10	611	778	136.3	63.2	73.2
	15	646	852	153.4	69.2	84.2

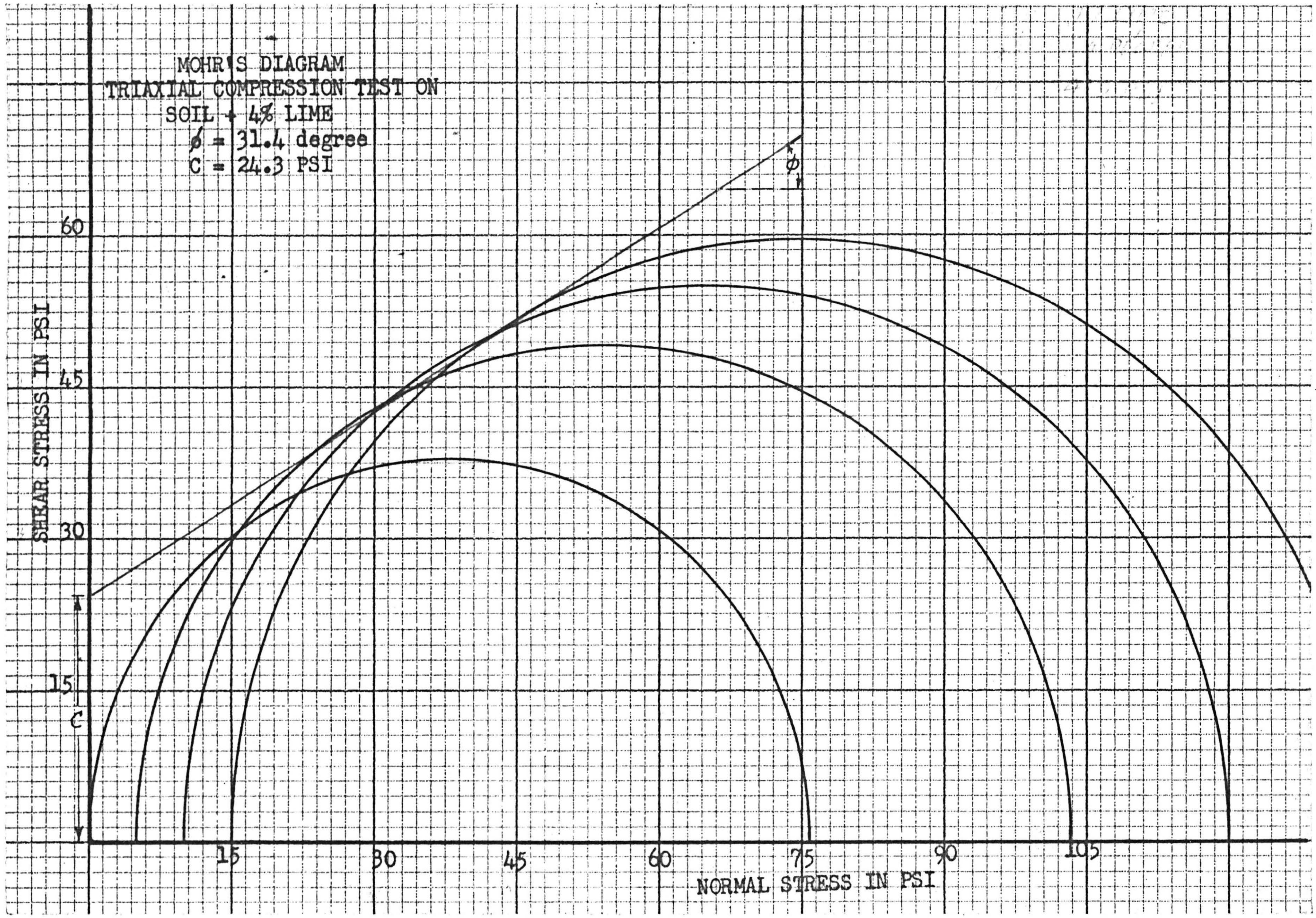
*Ultimate Stress = $\frac{\text{load at failure}}{\text{area}}$ + lateral pressure; area = 6.158 sq. in.

MIXTURE	LA ERAL PRESSURE σ_1 P.S.I.	AVERAGE DIAL READING AT FAILURE 10^{-4} IN.	LOAD AT FAILURE LBS.	ULTIMATE STRESS σ_3 P.S.I.	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\sigma_1 + \sigma_3}{2}$
Soil + 8% Lime	0	533	612	99.4	49.7	49.7
	5	581	715	21.1	58.1	63.1
	10	619	795	139.1	64.6	74.6
	15	657	875	157.1	71.1	86.1
Soil + 10% Lime	0	539	625	101.5	50.8	50.8
	5	570	692	117.4	56.2	61.2
	10	631	821	143.3	66.7	76.7
	15	672	906	156.1	73.6	88.6
Soil + 2% Cement	0	390	360	58.5	29.3	29.3
	5	441	420	73.2	34.1	39.1
	10	465	469	86.2	38.1	48.1
	15	488	518	99.1	42.1	57.1
Soil + 4% Cement	0	444	425	69.0	34.5	34.5
	5	502	548	93.8	44.4	49.4
	10	536	620	110.7	50.4	60.4
	15	558	666	123.2	54.1	69.1

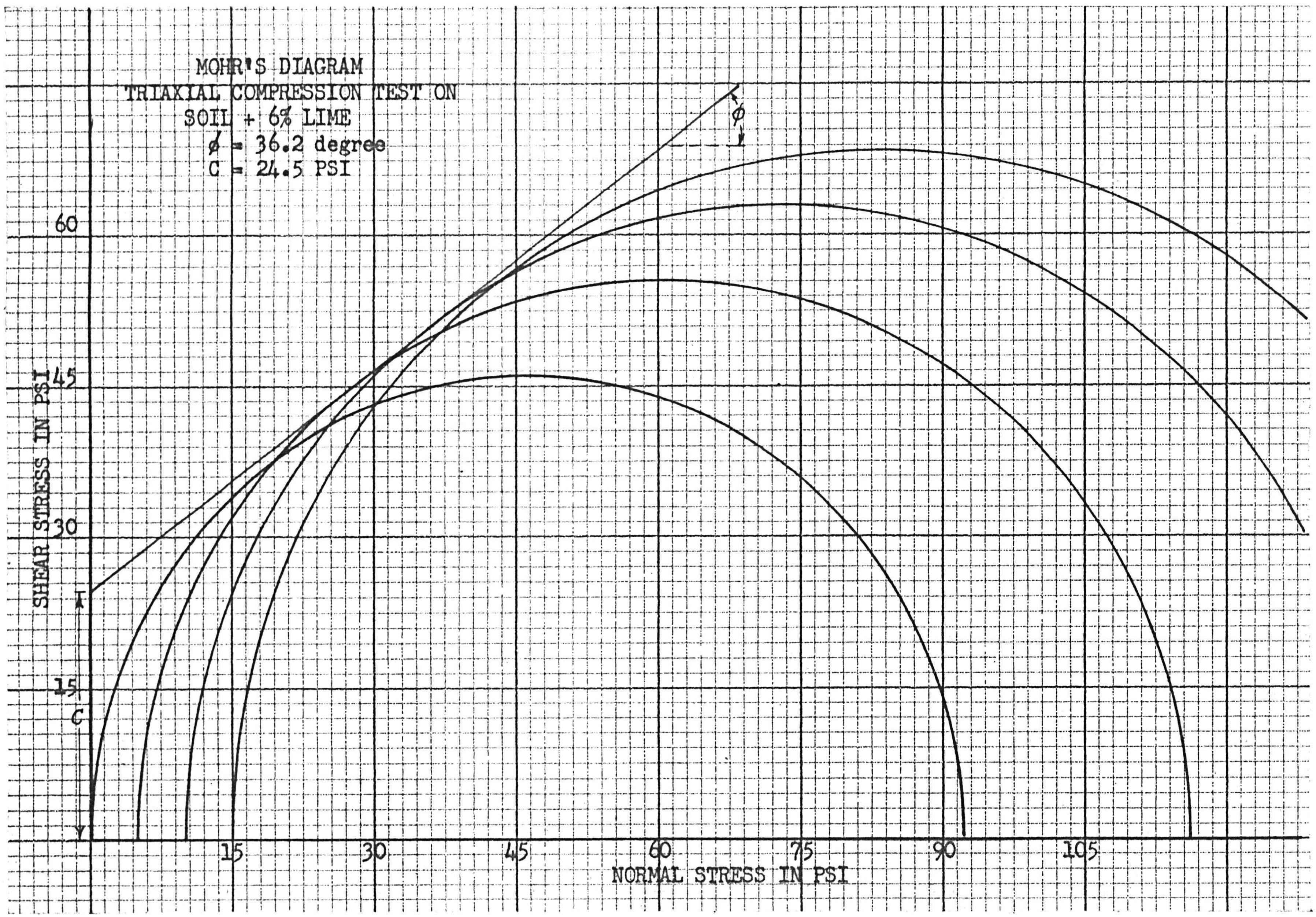
MIXTURE	LATERAL PRESSURE σ_1 P.S.I.	AVERAGE DIAL READING AT FAILURE 10^{-4} IN.	OAD AT FAILURE LBS.	ULTIMATE STRESS σ_3 P.S.I.	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\sigma_1 + \sigma_3}{2}$
Soil + 6% Cement	0	532	610	99.1	49.6	49.6
	5	577	707	119 8	57.4	62.4
	10	610	776	136.0	63.0	73.0
	15	634	826	149.1	67.1	82.1
Soil + 8% Cement	0	562	674	109.5	54.8	54.8
	5	598	750	126.8	60.9	65.9
	10	612	780	136.7	63.4	73.4
	15	641	842	151.7	68.4	83.4
Soil + 10% Cement	0	553	655	106.4	53.2	53.2
	5	586	726	122.9	58.9	63.9
	10	620	797	139.4	64.7	74.7
	15	647	854	153.7	64.4	84.4


MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON SOIL
ø = 16.6 degree
C = 17 PSI
SHEAR STRESS IN PSI
NORMAL STRESS IN PSI
60
45
30
15
C
ø
15
30
45
60
75
90
105

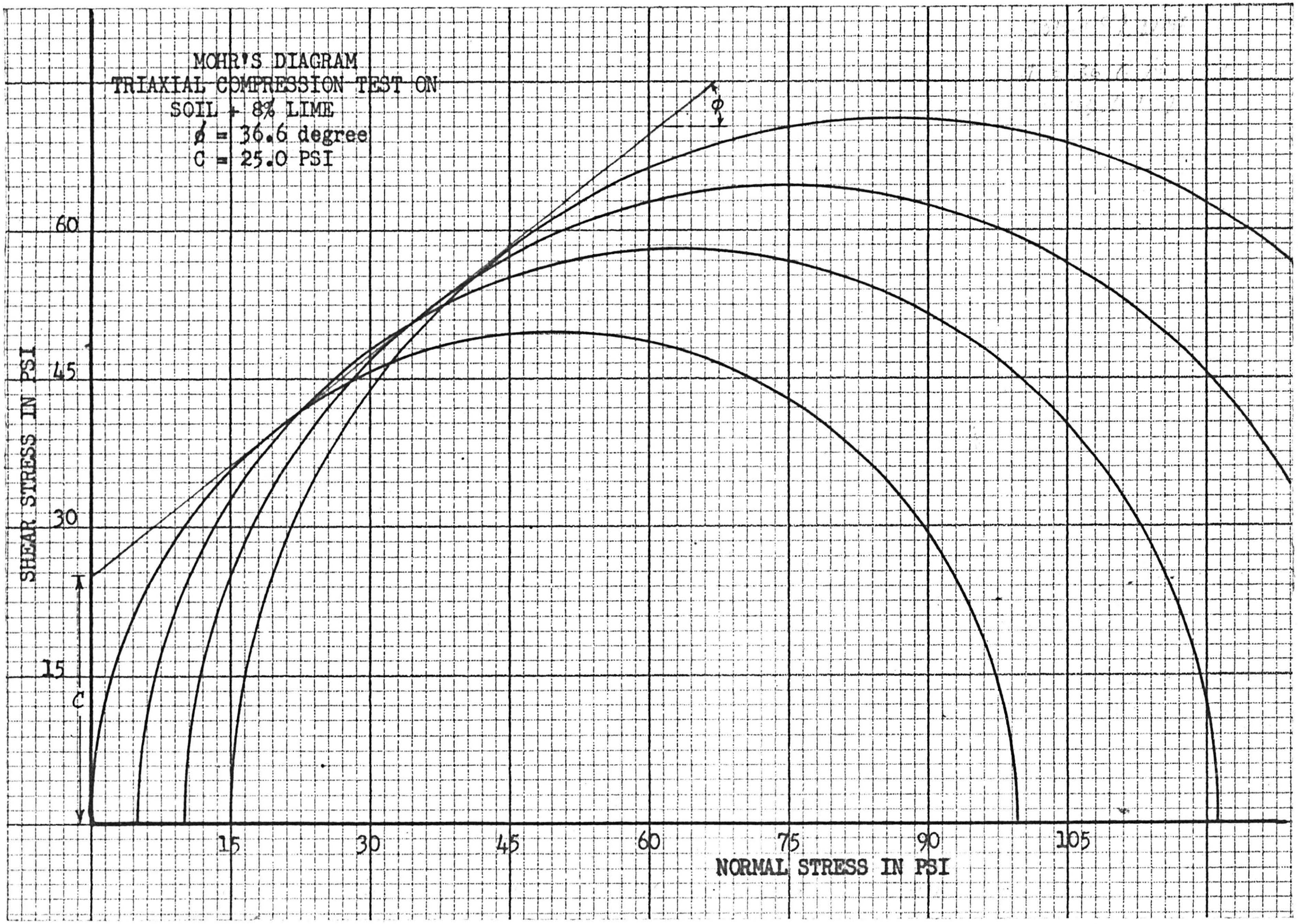

MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 2% LIME
ϕ = 23.9 Degree
C = 21.5 PSI
SHEAR STRESS IN PSI
60
45
30
15
C
ϕ
15
30
45
60
75
90
105
NORMAL STRESS IN PSI



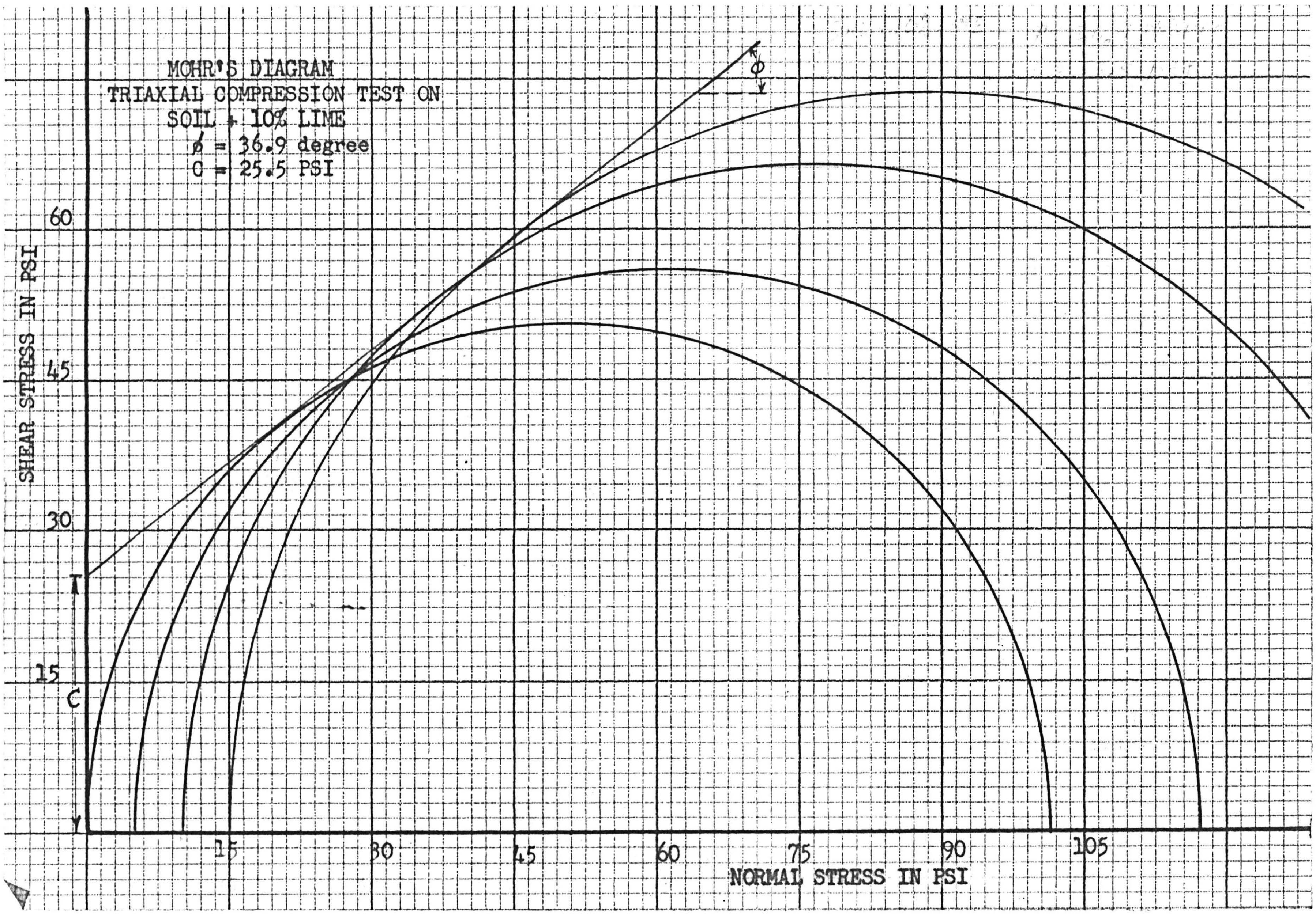
MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 4% LIME
ø = 31.4 degree
C = 24.3 PSI
ø
C
SHEAR STRESS IN PSI
60
45
30
15
15
30
45
60
75
90
105
NORMAL STRESS IN PSI

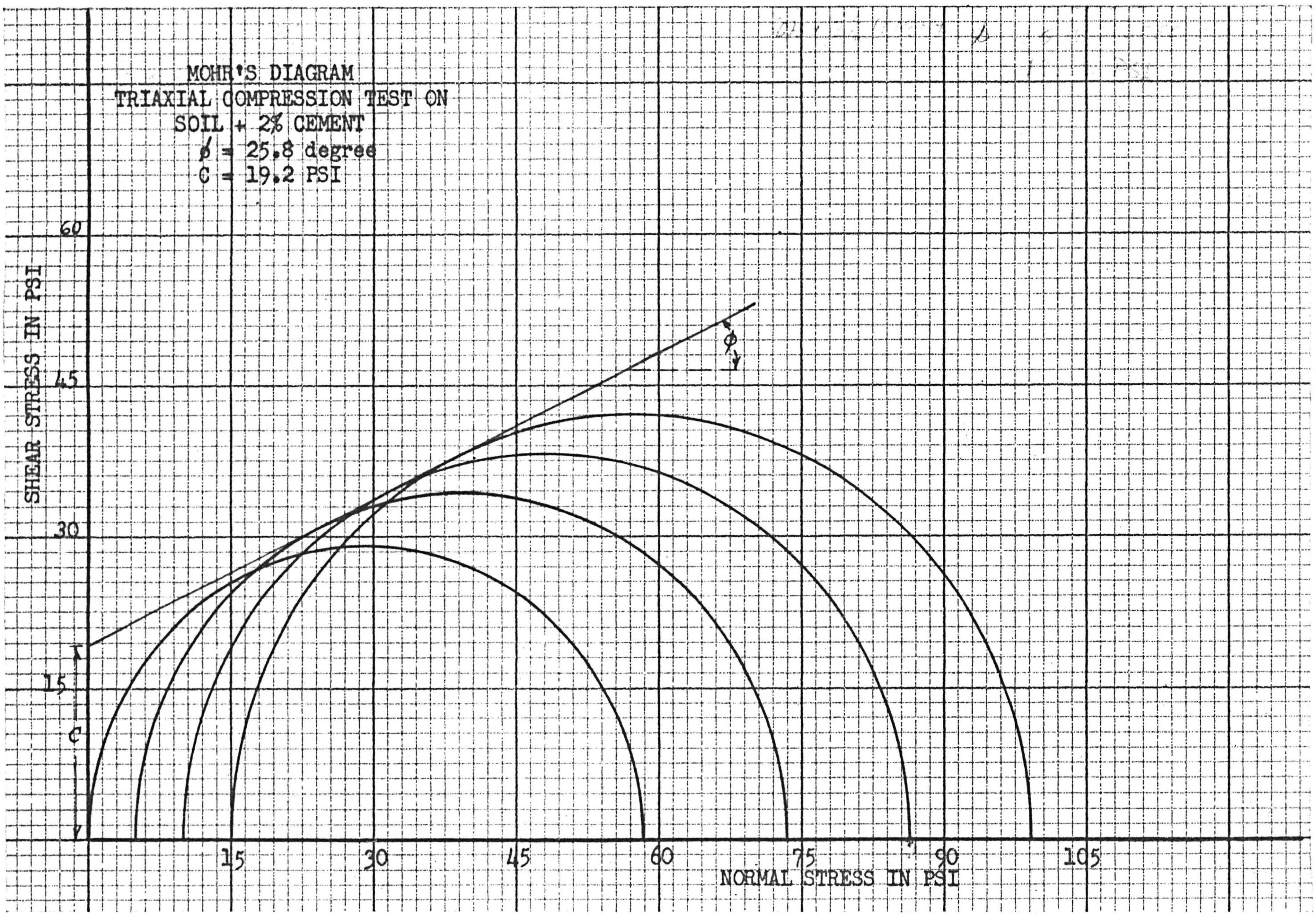


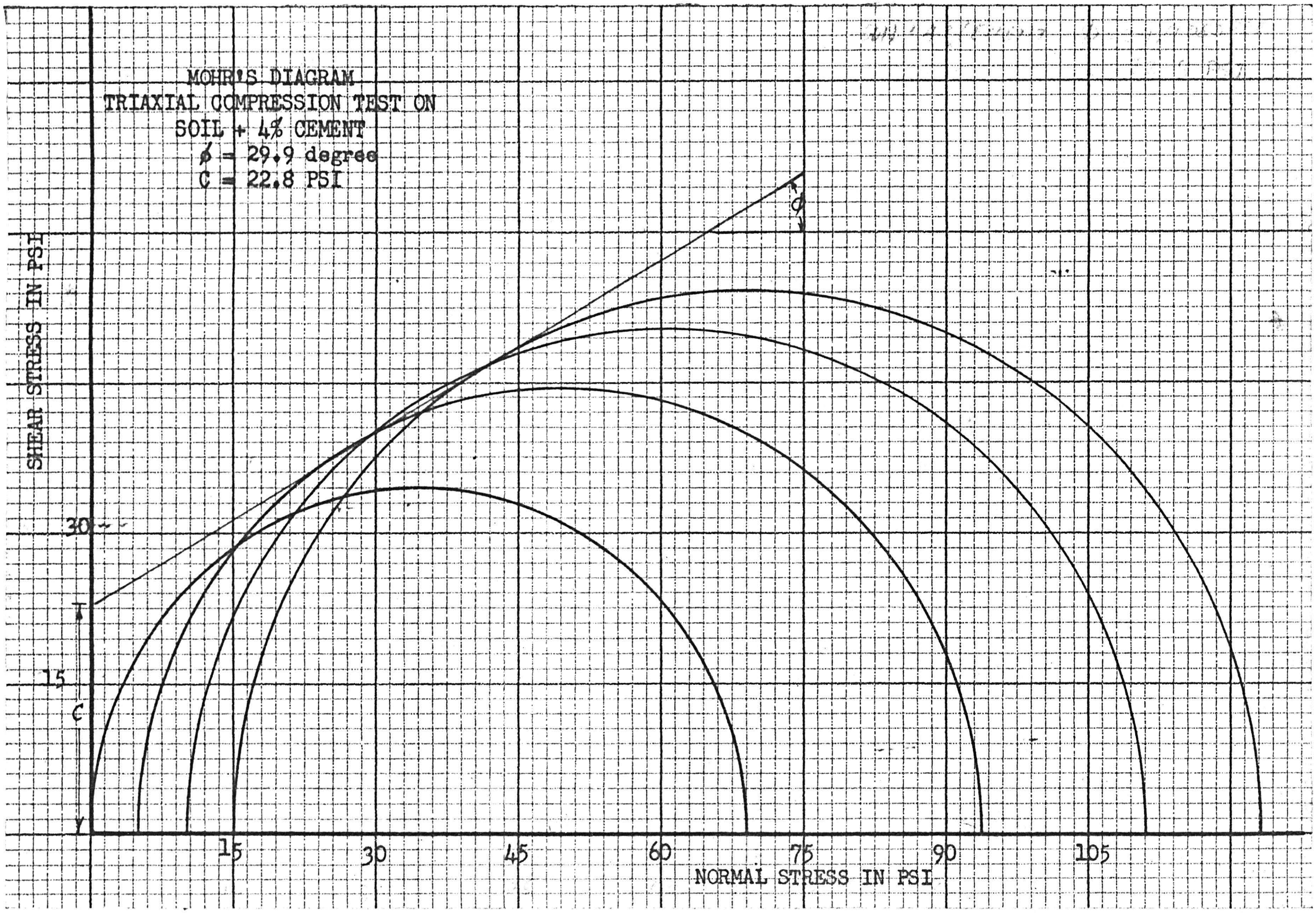
MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 6% LIME
ϕ = 36.2 degree
C = 24.5 PSI
ϕ
60
45
30
15
C
SHEAR STRESS IN PSI
15
30
45
60
75
90
105
NORMAL STRESS IN PSI



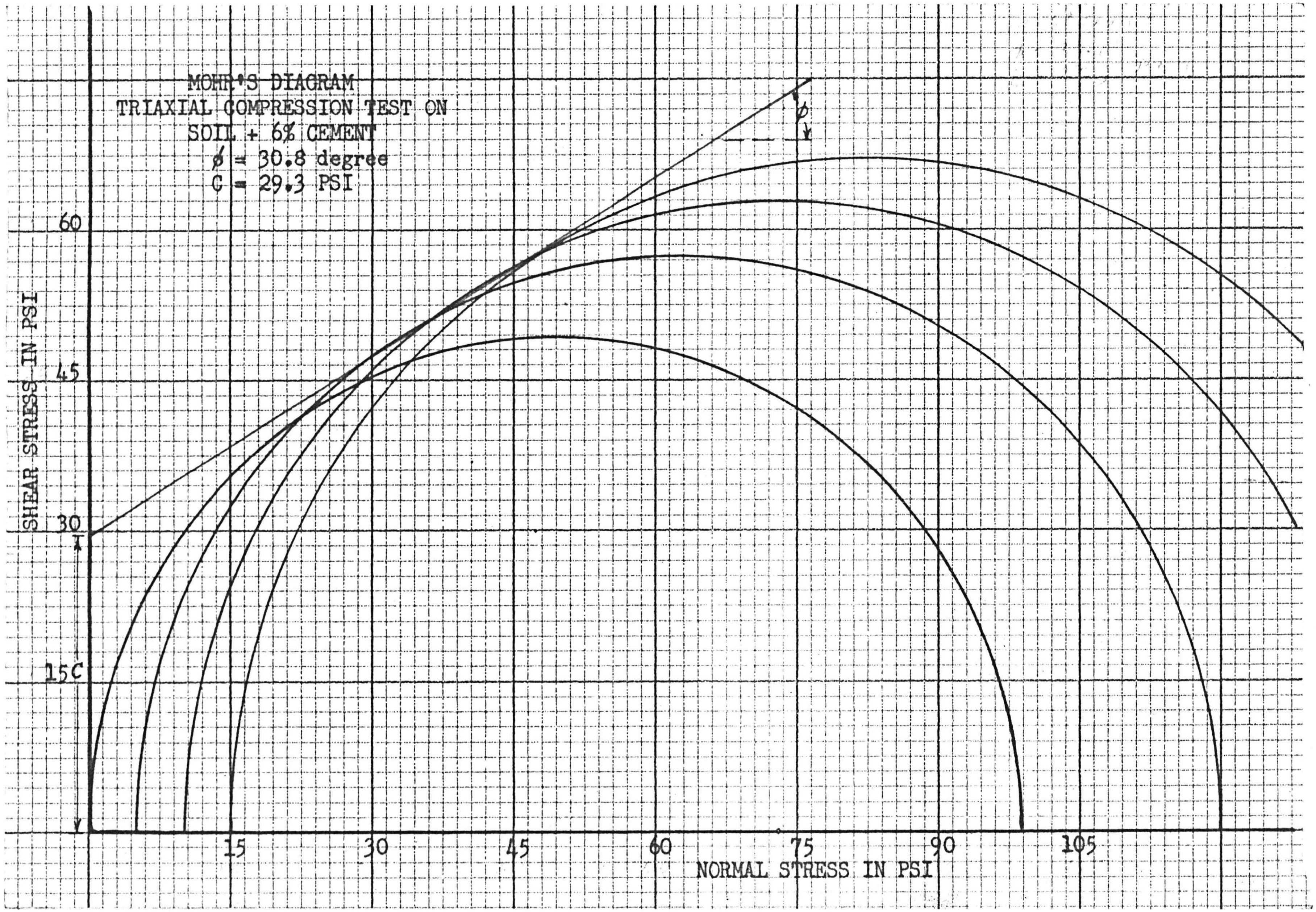
MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 8% LIME
ϕ = 36.6 degree
C = 25.0 PSI
ϕ
SHEAR STRESS IN PSI
60
45
30
15
C
15
30
45
60
75
90
105
NORMAL STRESS IN PSI

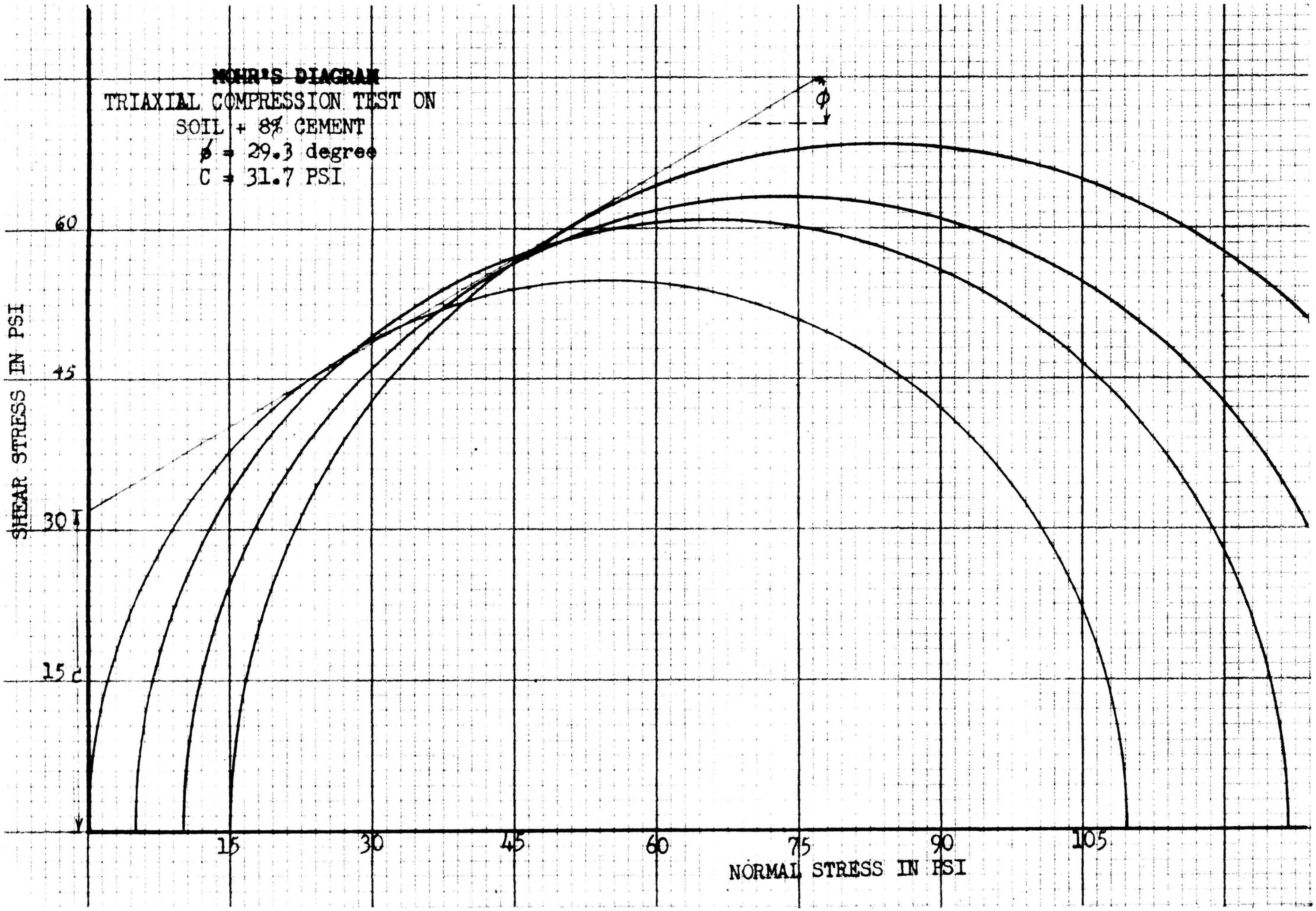

MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 10% LIME
ϕ = 36.9 degree
C = 25.5 PSI
ϕ
C
SHEAR STRESS IN PSI
15
30
45
60
NORMAL STRESS IN PSI
15
30
45
60
75
90
105


MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 2% CEMENT
ϕ = 25.8 degree
C = 19.2 PSI
SHEAR STRESS IN PSI
NORMAL STRESS IN PSI
60
45
30
15
C
ϕ
15
30
45
60
75
90
105

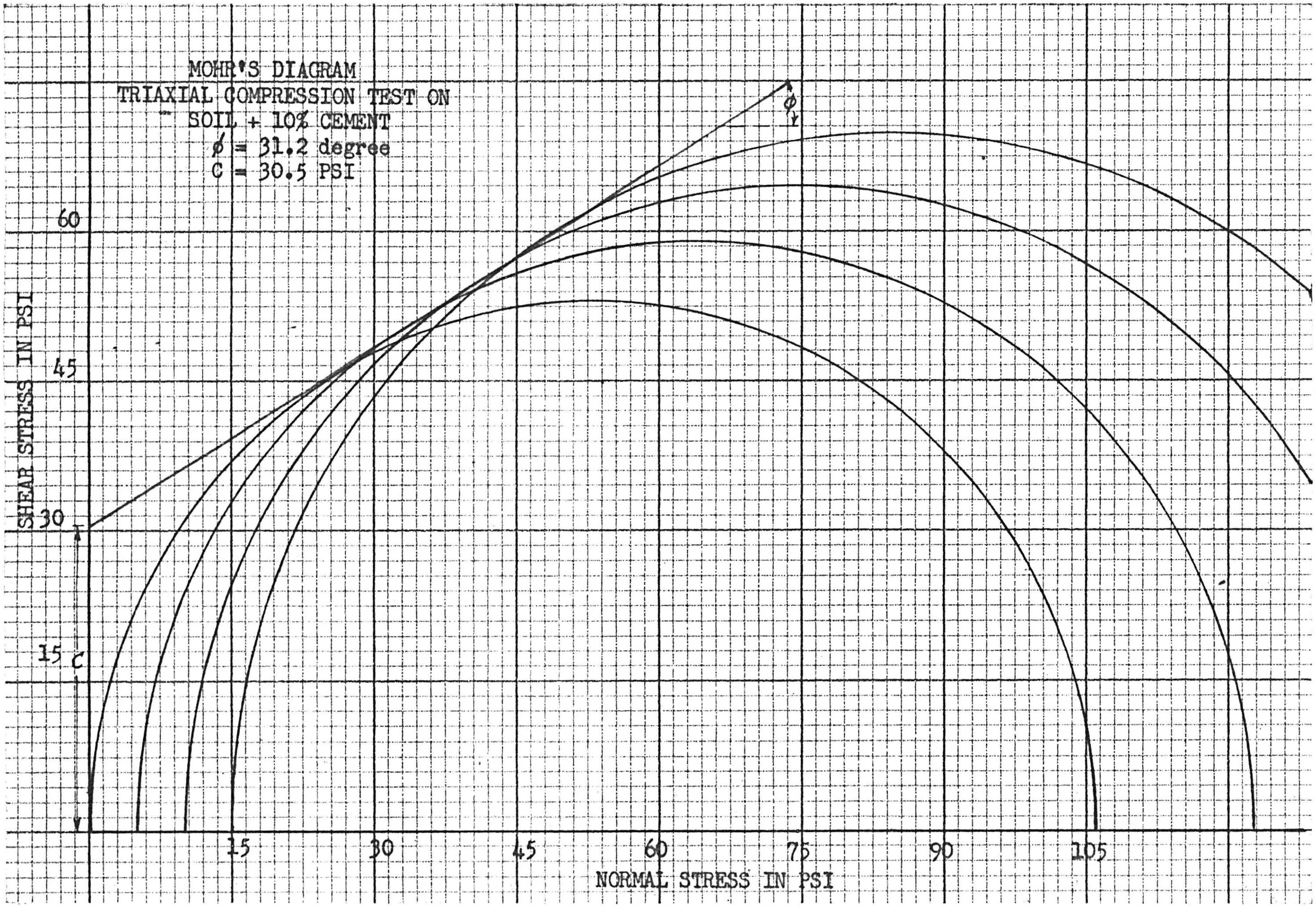


MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 4% CEMENT
ϕ = 29.9 degree
C = 22.8 PSI
SHEAR STRESS IN PSI
NORMAL STRESS IN PSI
15
30
45
60
75
90
105
C
ϕ


MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 6% CEMENT
ø = 30.8 degree
C = 29.3 PSI
ø
60
45
30
15
C
SHEAR STRESS IN PSI
15
30
45
60
75
90
105
NORMAL STRESS IN PSI



MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 8% CEMENT
ø = 29.3 degree
C = 31.7 PSI
ø
60
45
30
15
c
SHEAR STRESS IN PSI
15
30
45
60
75
90
105
NORMAL STRESS IN PSI


MOHR'S DIAGRAM
TRIAXIAL COMPRESSION TEST ON
SOIL + 10% CEMENT
ϕ = 31.2 degree
C = 30.5 PSI
ϕ
C
SHEAR STRESS IN PSI
15
30
45
60
NORMAL STRESS IN PSI
15
30
45
60
75
90
105

BIBLIOGRAPHY

1. Book:

Spangler, M. G., Soil Engineering, Scranton, Pennsylvania, International, 1951, p. 29.

2. Publications of Learned Societies:

D854-52, Specific Gravity of Soils, A.S.T.M. Standards, Part 3, pp. 1786-1788, 1955.

D423-54T, Liquid Limit of Soils, A.S.T.M. Standards, Part 3, pp. 1769-1773, 1955.

D424-54T, Plasticity Index of Soils, A.S.T.M. Standards, Part 3, pp. 1774-1776, 1955.

D422-54T, Grain Size Analysis of Soils, A.S.T.M. Standards, Part 3, pp. 1756-1766, 1955.

D429-39, Shrinkage Factors of Soils, A.S.T.M. Standards, Part 3, pp. 1782-1785, 1955.

D698-42T, Moisture-Density Relations of Soils, A.S.T.M. Standards, Part 3, pp. 1789-1791, 1955.

D424-54T, Plastic Limit and Plasticity Index of Soils, A.S.T.M. Standards, Part 3, pp, 1774-1776.

Classification of Highway Subgrade Materials, Standard Specifications for Highway Materials and Methods of Sampling and Testing, A.A.S.H.O., Part 1, pp. 45-51, 1955.

The PCA Soil Primer, Portland Cement Association, p. 41.

Johnson, A. M., Laboratory Experiments with Lime-Soil Mixtures, Proceedings, Highway Research Board, Vol. 28, pp. 496-507, 1948.

Whitehurst, E. A., and Yoder, E. J., Experiments of Durability of Lime-Stabilized Soil, Proceedings, Highway Research Board, Vol. 31, pp. 529-540, 1952.

Mills, W. H., Stabilizing Soils with Portland Cement, Proceedings, Highway Research Board, Vol. 16, pp. 322-349, 1936.

Catton, M. D., Research on the Physical Relations of Soil and Soil-Cement Mixtures, Proceedings, Highway Research Board, Vol. 20, pp. 821-855, 1940.

Felt, E. J., Factors Influencing Physical Properties of Soil-Cement Mixtures, Highway Research Board, Bulletin No. 108, pp. 138-162, 1955.

Dawson, R. F., Special Factors in Lime Stabilization, Highway Research Board, Bulletin 129, pp. 103-110, 1956.

McDowell, C. and Moore, W. H., Improvement of Highway Subgrades and Flexible Bases by the Use of Hydrated Lime, Proceedings of the Second International Conference on Soil Mechanics and Foundation, Vol. 5, pp. 260-267, 1948.

Gallaway, B. M. and Buchanan, S. J., Lime Stabilization of Clay Soil, Texas Engineering Experiment Station, Bulletin No. 124, 1951.

3. Periodicals:

McCaustland, E. J., Lime in Dirt Road, Pit and Quarry, Vol. 10, No. 5, pp. 93-95, June 1, 1925.

Mills, W. H., Road Base Stabilization with Portland Cement, Engineering News-Record, Vol. 115, pp. 751-753, November 28, 1935.

Sheets, F. T. and Catton, M. D., Basic Principles of Soil-Cement Mixtures, Engineering News Record, Vol. 120, pp. 869-875, June 23, 1938.

4. Pamphlet:

Woods, K. B., Lime as an Admixture for Base and Subgrades, Paper presented at the 31st Annual Convention of the National Lime Association, 1949.

VITA

Judson Leong was born on October 28, 1930, at Mergui, Burma. His college education began at the University of Rangoon, Rangoon, Burma in 1948. In 1950, he was admitted into the Engineering College of the University of Rangoon.

He came to the United States and enrolled at the Missouri School of Mines in February, 1952. After receiving a B. S. Degree in Mining Engineering in January, 1954 and a B. S. Degree in Civil Engineering in June, 1955, he was employed as a Civil Engineer by the Illinois Division of Highways.

He enrolled for graduate work at the Missouri School of Mines in September, 1956, with the appointment of Graduate Assistant in the Civil Engineering Department.

He is a Junior Member of the American Institute of Mining and Metallurgical Engineers; American Society of Civil Engineers and a member of Chi Epsilon, Civil Engineering Honor Fraternity.